# Final Report

## Jetty Road Rezoning – Stage 2 SWMS

Curlewis Bellarine Pty Ltd

15 February 2023

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## Project Details

| | |
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| **Project Name** | Jetty Road Rezoning – Stage 2 SWMS |
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# 1 INTRODUCTION

The following report sets out a recommended Stormwater Management Strategy (SWMS) for the proposed residential subdivision consisting of multiple parcels known as Jetty Road Stage. This report forms part of the Development Plan Overlay (DPO) application for Jetty Road – Stage 2.


**FIGURE 1-1 JETTY ROAD URBAN GROWTH AREA (SOURCE: JETTY ROAD URBAN GROWTH AREA DCP 2011)**

## 1.1 Objective

The key objective of the SWMS is to identify the key drainage and Water Sensitive Urban Design (WSUD) assets and their sizing to ensure that the peak discharge rate, volume and pollutant load of stormwater leaving the development area are no greater than pre-development rates. This has been undertaken in accordance with the flooding and stormwater drainage management guiding principles of the Jetty Road Urban Growth Plan (UGP)[1] and in line with both City of Greater Geelong (the City) and Corangamite Catchment Management Authority (CCMA) surface water management requirements.

[1] City of Greater Geelong (September 2008), Jetty Road Urban Growth Plan. Available at https://www.geelongaustralia.com.au/common/Public/Documents/8d071224bda1b1f-Jetty%20Road%20UGP%20-%20Amended%2023%20Sept%202008.pdf

# 2 BACKGROUND

The Jetty Road Stage 2 land is part of the Jetty Road Urban Growth Area, located within the Clifton Springs/Drysdale Catchment Management Unit areas[2] shown in Figure 2-1. Managing flooding risk is the focus for the City in this catchment area. The Jetty Road Urban Growth Plan[1] provides a general stormwater catchment plan which indicates the southern portion of the Jetty Road Stage 2 land drains in a north-westerly direction to a waterway which crosses Coriyule Road and Scarborough Road. The northern portion predominately drains north towards Port Philip Bay and a small portion of the northern land drains east towards Griggs Creek (Figure 2-1).

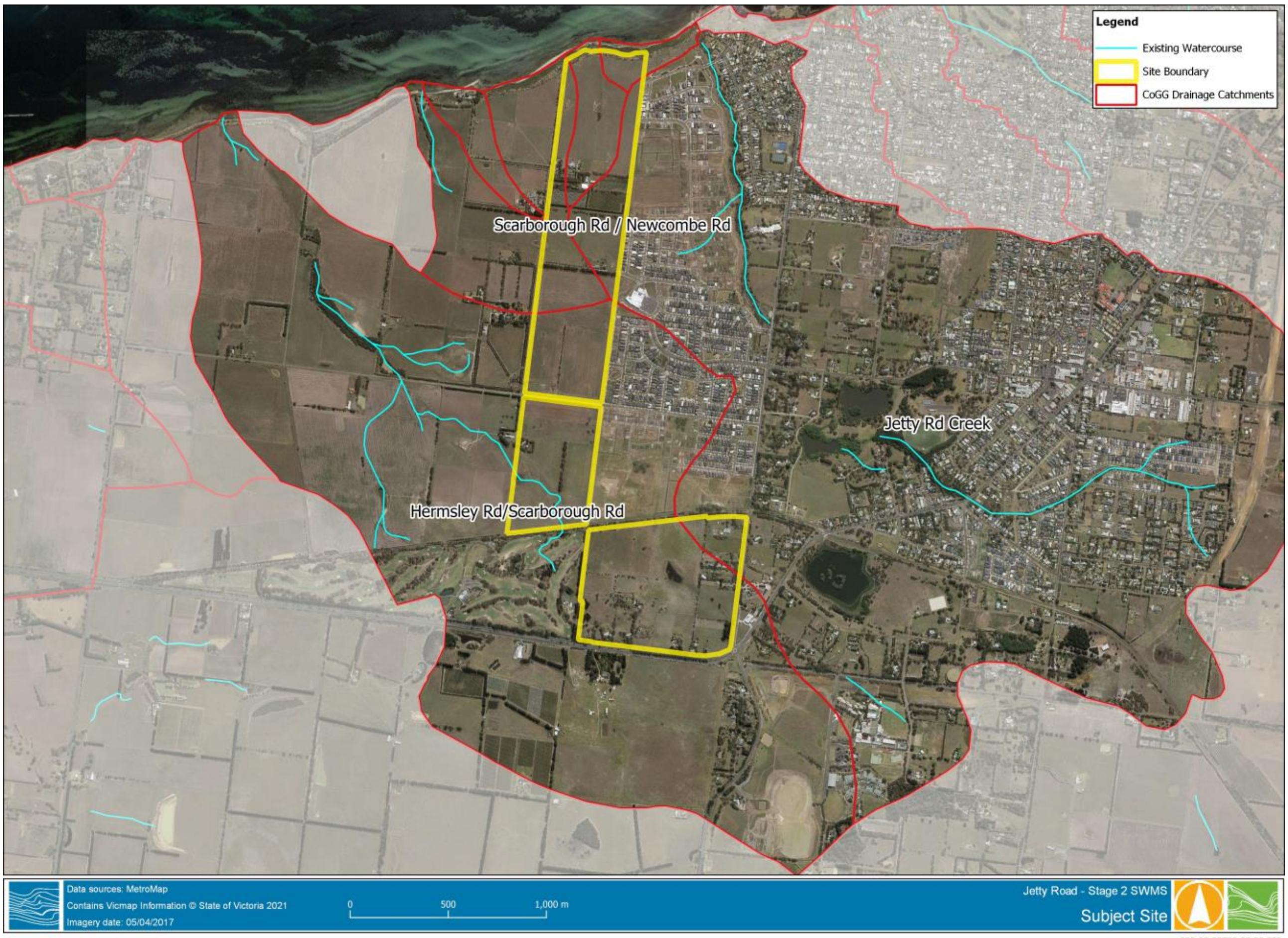


**FIGURE 2-1 SUBJECT SITE**

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[2] City of Greater Geelong Stormwater Management Strategy 2020-30, published June, 2020

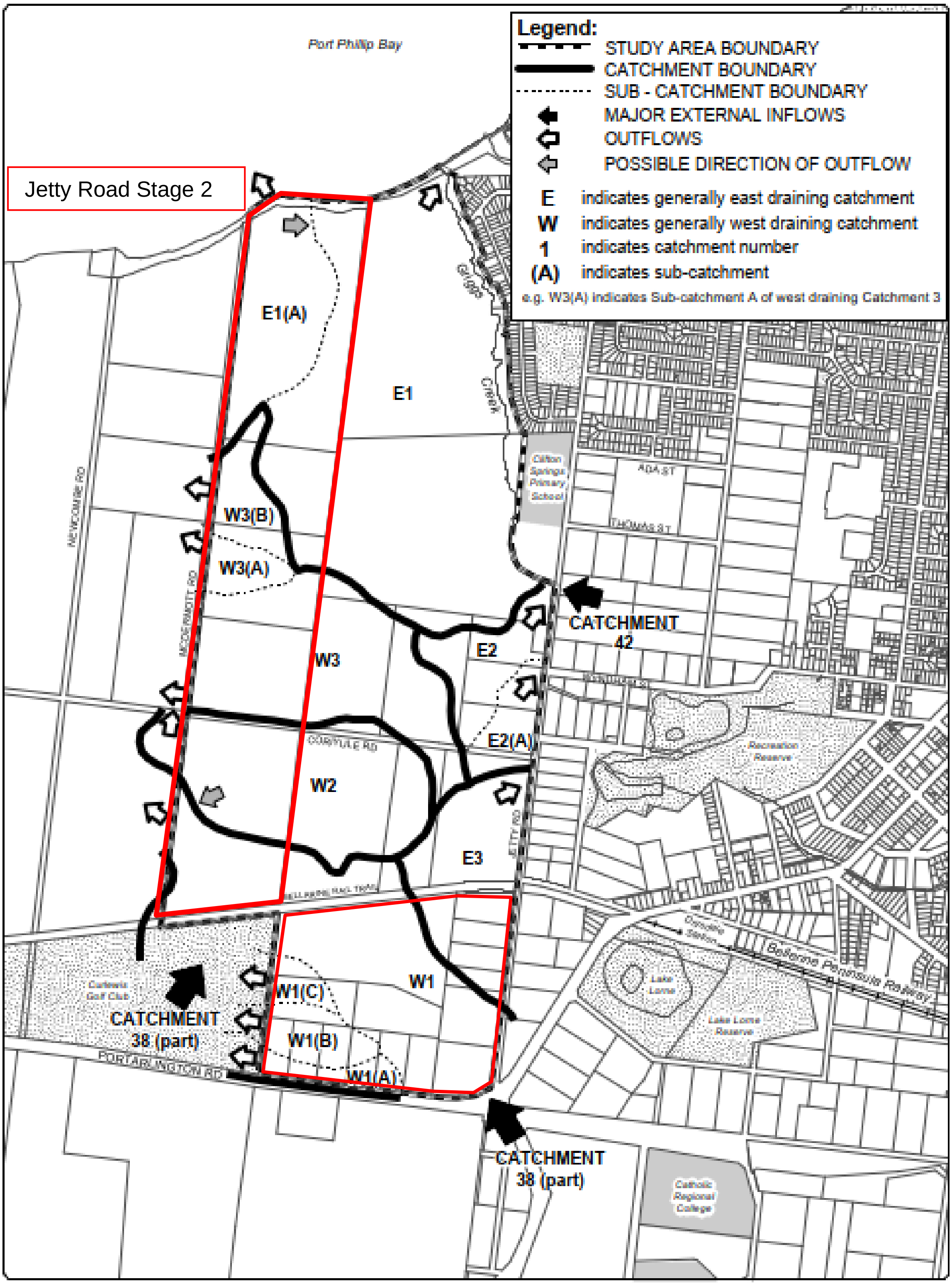


FIGURE 2-2 STORMWATER CATCHMENT PLAN (SOURCE: MAP 16 JETTY ROAD URBAN GROWTH PLAN)

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## 2.1 Proposed Development

Jetty Road Stage 2 encompasses 30 properties and an area of approximately 150 ha. The latest Master Plan of the proposed development is shown in Figure 2-3 and is planned to include approximately 1600 new residential lots.

It is noted that the proposed Master Plan may change as the development progresses. Provided that the overall density and layout are not significantly altered, minor revisions will not impact the overall drainage and water quality concept design presented in this report.

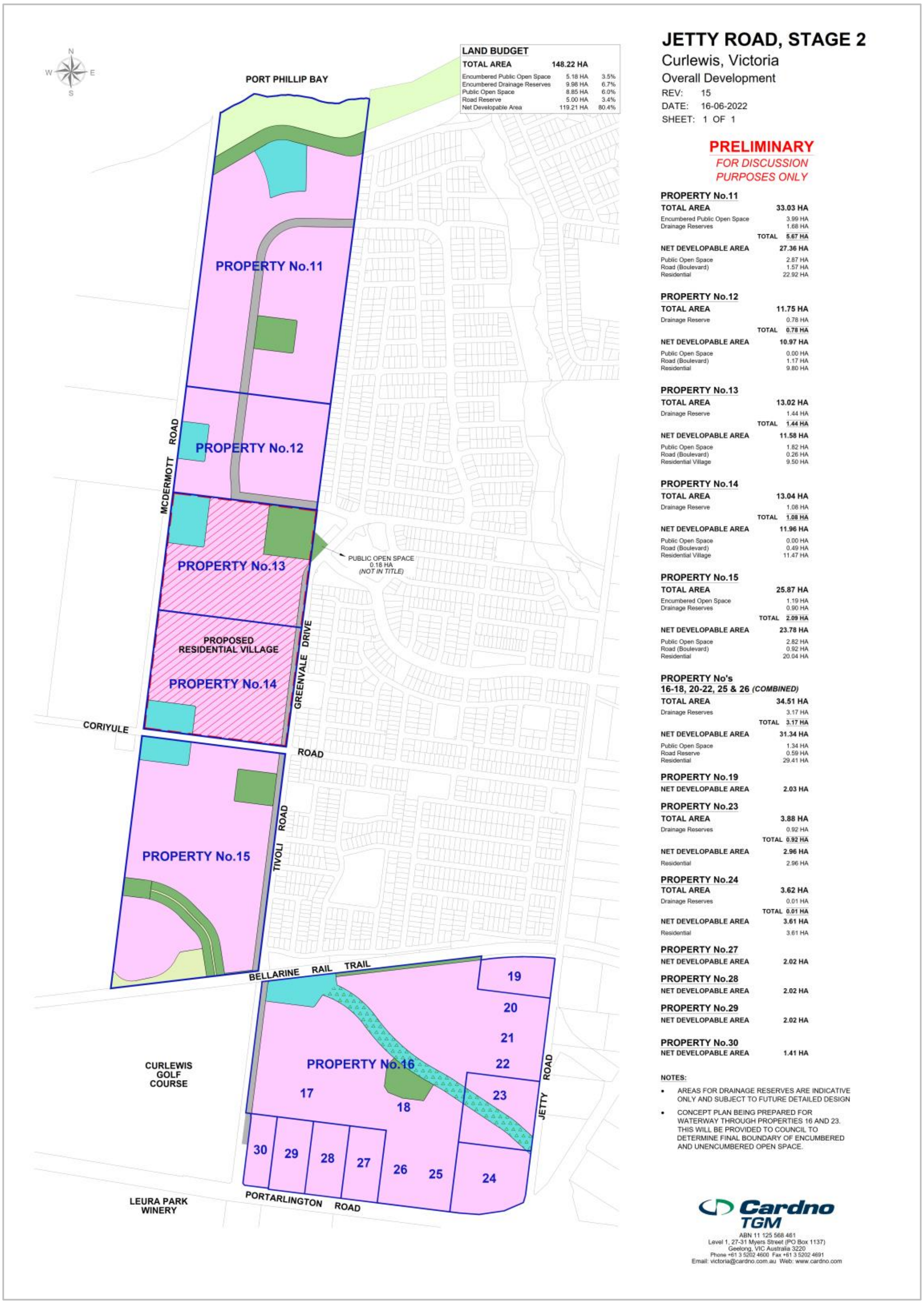


**FIGURE 2-3 DEVELOPMENT CONCEPT PLAN (SUPPLIED BY CARDNO TGM, JUNE 2022)**

## 2.2 Existing Waterways

There is a single designated waterway within the site (Figure 2-1). The CCMA requires for this waterway (Designated Waterway ID 33-30) to be protected, and where possible, enhanced as part of the residential development. The following advice was provided by the CCMA (CCMA-F-2019-01057) in January 2020 and again in February 2022:

- Corangamite CMA has noticed an increase in developments that propose to pipe waterways, usually for the purpose of allowing low flows in the waterway to bypass urban stormwater treatment assets. The CMA is opposed to this practice and will not support the granting of a permit that includes such a proposal. Furthermore, the CMA will not grant a Works on Waterways Permit for such works.
- The waterways will require a setback of at least 20 metres from each bank, in accordance Melbourne Water's *Waterway Corridors: Guidelines for greenfield development areas within the Port Philip and Westernport Region*; and
- Any works within, above or below the bed and banks of a designated waterway require a Works on Waterways Permit from the CMA prior to commencement.

Details regarding the proposed constructed waterway which has been design in accordance with Melbourne Water (2013) Waterway Corridor guidelines are provided in Section 3.7.

**FIGURE 2-4 DESIGNATED WATERWAYS (SOURCE CCMA FLOOD ADVICE)**

# 3 STORMWATER MANAGEMENT

This section of the SWMS details proposed internal drainage infrastructure within Jetty Road Stage 2. The objective is to guide drainage design for the site to provide for the collection, treatment and disposal of stormwater runoff in an environmentally acceptable manner within the subdivision layout, consistent with applicable guidelines and standards and including the implementation of best practice water quality measures.

## 3.1 Legal Points of Discharge

Under existing conditions, approximately 126 ha of the site drains in a westerly direction with the remaining ~24 ha (located at the north of the site) drains east towards Griggs Creek. The current Master Plan has identified seven main outfalls (located along the western boundaries of several properties). It was assumed that under the proposed development conditions, the overall site level strategy will be set in a way that all of the easterly draining catchment will drain west to the identified outfalls.

The proposed outfall locations are shown in Figure 3-1. Outfalls 1, 2, 5, 6 and 7 and considered free draining whereas Outfalls 3 and 4 are to be connected to the existing Council drainage network (piped) along Coriyule Road. However, as it is a requirement of the stormwater management plan to ensure the post-development runoff volume do not exceed the pre-development runoff volume, Outfalls 2, 5 and 6 which are discharging to downstream properties will have an additional diversion pipe directing some of the runoff towards ocean outfalls (Outfalls 3, 4 and 7) as shown in Figure 3-1.

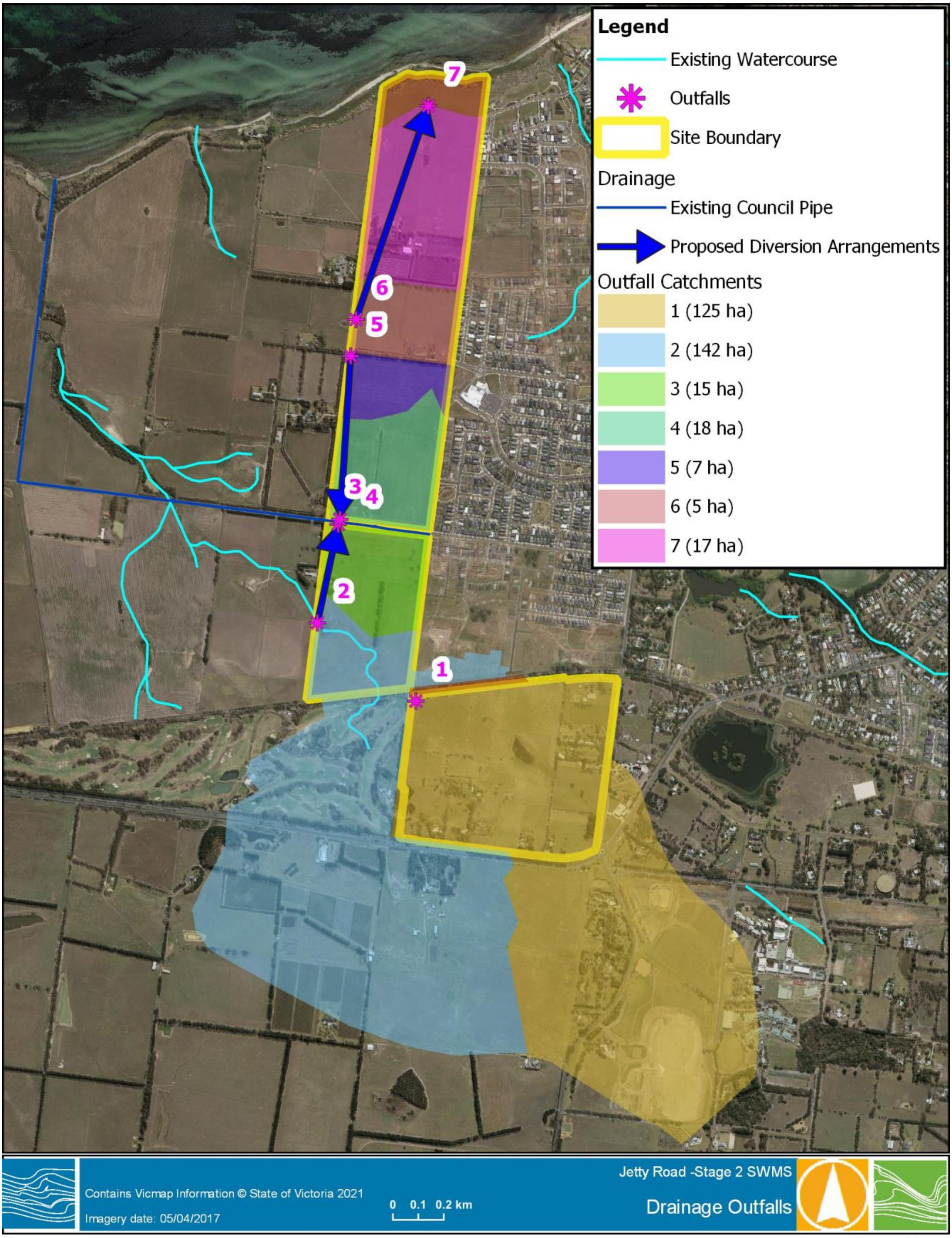


FIGURE 3-1 DRAINAGE OUTFALLS

## 3.2 Hydrological Analysis

A rainfall-runoff routing model (RORB) was used as the principal tool for the hydrologic analysis of this study area. RORB is a non-linear rainfall runoff and stream flow routing model for calculation of flow hydrographs in drainage and stream networks. The model was developed in accordance with the latest Australian Rainfall and Runoff (ARR2019) rainfall datasets and guidelines. This RORB modelling assesses the impacts of the proposed site development on the peak 1% AEP (Annual Exceedance Probability) flow leaving the site and associated volume of runoff.

Model scenarios were developed and simulated for, pre-developed (100% rural), existing and development conditions to determine the flooding mechanisms across the site and to size the retarding basins and outlet configurations. Details of the RORB modelling are provided in the following sections.

### 3.2.1 Pre-developed Conditions

No streamflow data exists within the catchment to undertake a calibration of the hydrology model. To identify appropriate model parameters, a validation process through comparison with the Regional Flood Frequency Estimate (RFFE) tool was undertaken. The pre-developed conditions model represents a 100% rural/natural catchment with no hydraulic controls. This scenario was modelled to validate parameters including kc (RORB routing parameter) and loss values against the RFFE tool.

Sub-catchments were delineated using 2012/13 LiDAR data. The total catchment area was estimated as 16.1 $km^2$. The catchment plan of the existing conditions RORB model is shown in (Figure 3-2). The Fraction Impervious (FI) value of all sub-catchments was set to zero and all reaches in RORB model was modelled as natural to reflect the 100% rural/natural conditions for the catchment.

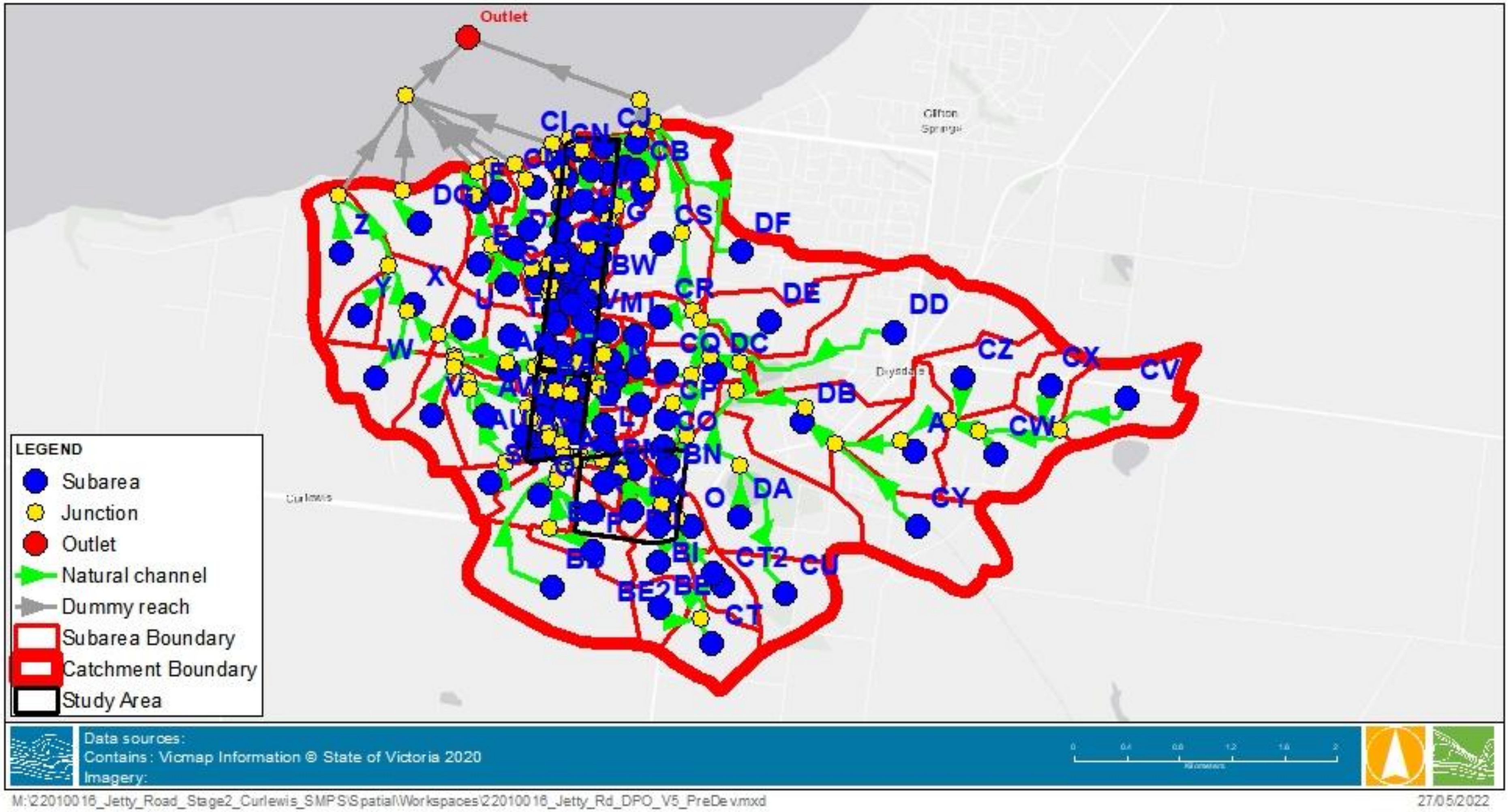


**FIGURE 3-2 PRE-DEVELOPED CONDITION RORB CATCHMENT PLAN**

The following approach was adopted to determine the loses, routing parameters for RORB model validation:

- The initial and continuing losses as in Table 3-1 were used in the modelling, in accordance with ARR2019 (Book 5, Chapter 3.5.3). The pervious area losses were extracted from the ARR2019 Data Hub.

TABLE 3-1 INITIAL AND CONTINUING LOSSES

| | Initial Loss (mm) | Continuing Loss (mm/hr) |
|---|---|---|
| PA | 16.80 | 3.00 |

  - The Data Hub rural losses (19 mm and 3 mm/hour) were selected for the initial and continuing loss respectively for the catchment area. The initial loss from the Data Hub is for complete storms rather than bursts. As the rainfall modelled in RORB are bursts, rural initial losses are reduced by the pre-burst rainfall corresponding to the median pre-burst rainfall depth (mm) from the Data Hub. The median pre-burst rainfall depth (mm) varies across storm durations. 2.2 mm pre-burst rainfall depth was selected[3] and subtracted from the rural initial loss (19 mm) to derive the burst PA initial loss (19 mm – 2.2 mm = 16.8 mm).
  - The rural continuing loss (3 mm/hour) is based on a 1-hour timestep. In accordance with ARR 2019 Book 5, Chapter 3.5.3.2.2, continuous losses should typically range from 0-4 mm/h in south-eastern Australia. As a result, unmodified continuous loss of 3 mm/hour was used for PA.
- The Kc routing parameter value was derived using regional equations in accordance with the Melbourne Water Flood Mapping Projects Guidelines and Technical Specifications (July 2020)
- Total catchment area (A) was 16.08 $km^2$ and $d_{av}$ (average routing distance) was calculated as 2.94 km.
- The pre-developed conditions RORB model was run using the ARR2019 Intensity-Frequency-Duration (IFD) data and the selected Kc (routing parameter) values for the 1% AEP event across the ensemble of temporal patterns and a range of storm durations from 10 min to 72-hour.
- The 1% AEP median peak flow at the catchment outlet was compared against the RFFE (Appendix B).
- Equation No. 4 in Table 3-2 was chosen as the method for deriving the routing parameter as it provides the closest flow rate to the RFFE.

Table 3-2 Kc Values used in Pre-developed Conditions and comparison of flow estimates

| *Equation No.* | *Regional Equation* | *Kc* | *1% AEP Flow Estimate* |
|---|---|---|---|
| 1 | $k_c = 0.49 \times A^{0.65}$ | 2.98 | 33.5 |
| 3 | $k_c = 2.2 \times A^{0.5}$ | 8.82 | 15.2 |
| 4 | $k_c = 1.25 \times d_{av}$ | 3.68 | 26.9 |
| RFFE Estimate | | | 29.1 |

### 3.2.2 Existing Conditions

The existing conditions model represents the site under current conditions and provides maximum flow rates and flow volumes at each outfall. The catchment delineation of the existing conditions RORB model is shown in Figure 3-3 with a zoomed in version of the study area shown in Figure 3-4.

[3] The preburst depth of 2.2mm was selected as it is in line with critical duration of the overall catchment outlet used in model validation.

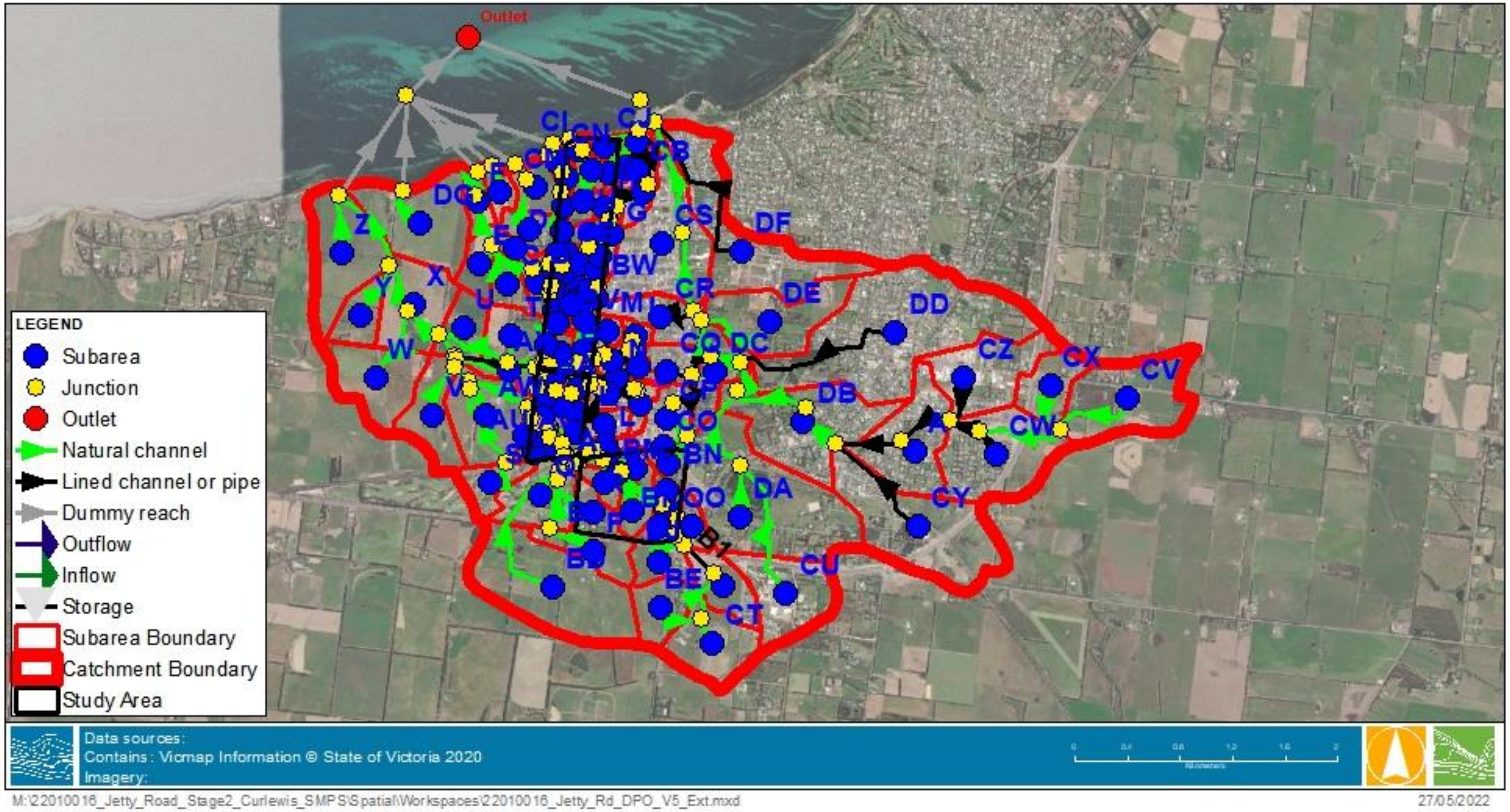


**FIGURE 3-3 EXISTING CONDITIONS RORB CATCHMENT PLAN (MODEL EXTENT)**

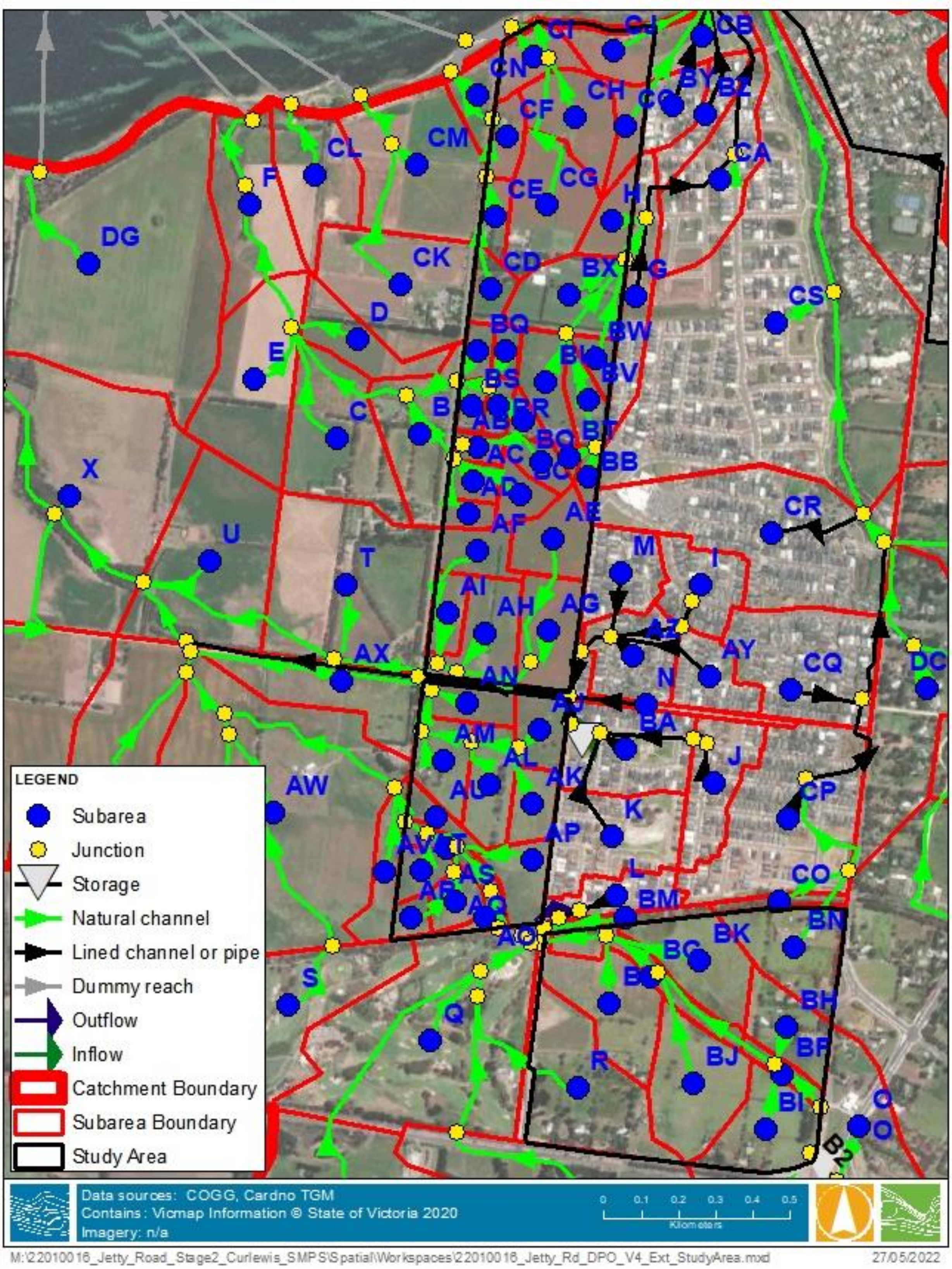


**FIGURE 3-4 EXISTING CONDITIONS RORB CATCHMENT PLAN (ZOOMED INTO STUDY AREA)**

To reflect the existing conditions of the site, the following approach was adopted:

- The initial and continuing losses as in Table 3-1 were used in the modelling, in accordance with ARR2019 (Book 5, Chapter 3.5.3). The pervious area losses were extracted from the ARR2019 Data Hub.

TABLE 3-3 INITIAL AND CONTINUING LOSSES

| | Initial Loss (mm) | Continuing Loss (mm/hr) |
|---|---|---|
| EIA | 1.50 | 0.00 |
| ICA | 13.30 | 2.50 |
| PA | 16.80 | 3.00 |

- The catchment impervious fractions for different surface types were determined. Firstly, Total Impervious Area (TIA) fraction values were determined for each land use type based on typical fraction impervious values outlined in the Melbourne Water MUSIC Guidelines (2018).
- Following this the catchment fractions were determined for the following three urban surface[4] types:
  - Effective Impervious Area (EIA)
    - EIA = 0.6 * TIA (ARR2016, Book 5, Chapter 3.4.2.2.2) when TIA ≤ 80%
    - EIA = 0.6 to 1.0 * TIA when TIA >80% as per EIA/TIA ration relationship presented in Figure 3-5.
    - EIA = 0 when in rural areas such as bushland, parklands where pervious area do not interact with impervious areas.

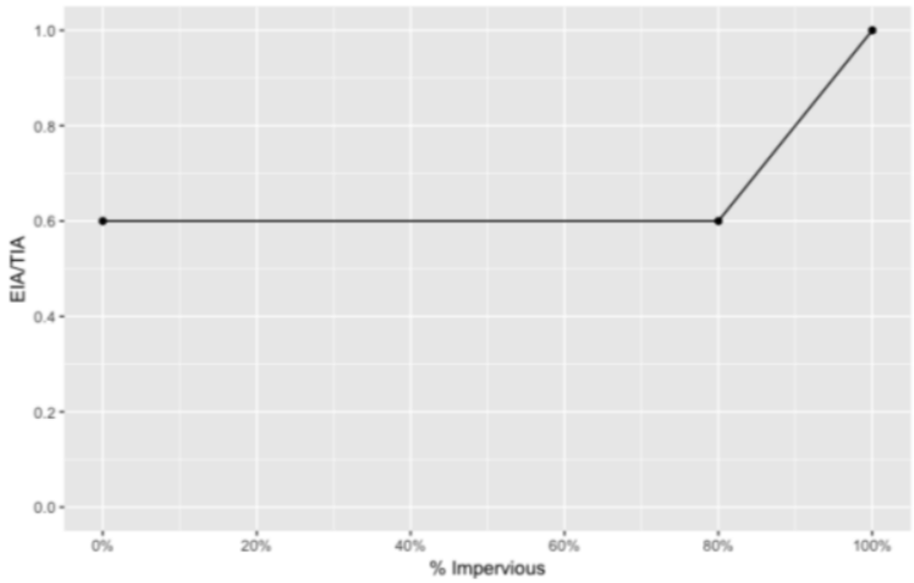


FIGURE 3-5 EIA/TIA RATIO INCREASE FOR HIGHLY IMPERVIOUS CATCHMENT

  - Pervious Area (PA)
    - PA = 1 – TIA
  - Indirectly Connected Area (ICA)

[4] Based on the Melbourne Water Flood Mapping Projects Guidelines and Technical Specifications (Melbourne Water, July 2020) and Australian Rainfall and Runoff 2019.

        - ICA = 1 – PA - EIA

- Five different reach types are available in RORB (1 = natural, 2= excavated & unlined, 3= lined channel or pipe, 4= drowned reach, 5= dummy reach). The reach types in the RORB model were set as followed:
    - "natural" for runoff being conveyed through non-formalised drainage paths (i.e., grassed slope without channel);
    - "excavated & unlined" for runoff being conveyed via open channel (grassed or earthen); and
    - "Lined channel or pipe" for runoff being conveyed from the residential lots subarea (roof/gutter/downpipes).
- Five existing retarding basins within the Bellaview Estate and Curlewis Parks Estate along with two upstream basins next to the of Drysdale bypass were included in the existing conditions RORB model.
    - The stage-storage-discharge relationships of the five basins within Bellaview Estate and Curlewis Parks Estate were derived using the basin parameters extracted from the existing conditions flood study (Cardno TGM, 2020)[5] XP-STORM model.
    - In absence of stage-storage information of the two basins controlling the flow from the external catchment upstream of the Outfall 1 (Drysdale Bypass Basins), aerial imagery and the Drysdale Bypass Report (MRPV, 2019[5]) were used to estimate the storage volume. The estimated volume of two basins (modelled as storages B1 and B2) was estimated as 59,000 $m^3$ and 18,000 $m^3$. The diameter of the outlet pipes of the two basins were set as 450 mm and 525 mm for B1 and B2 respectively (MRPV, 2019)[6].
        - A sensitivity check was conducted upon receiving the as-constructed plans provided by the Council. The 1% AEP flood storage of the basins B1 and B2 were modelled as 15,987 $m^3$ and 8,000 $m^3$ respectively. Outlet pipe lengths and slopes were also adjusted accordingly. Refer to Appendix A-3 for further details. A summary of modelling outcomes is presented below.
        - Modelling results indicate the basin B1(Grubbs Rd wetland/Retarding basin) flood storage exceeds the allocated volume at the 1% AEP event. It was assumed any runoff passing through the spillway would enter the downstream basin (B2)
        - Nevertheless, the 1% AEP peak outflow rate at the external catchment inflow at Jetty Road was estimated as 0.82 $m^3/s$ whereas the original model results indicated a peak flow rate of 0.81 $m^3/s$. Since the change in inflows are insignificant, original model results were considered accurate for the stormwater management plan development.
- The existing culvert crossing (1.3 m x 0.9 m) north of Curlewis golf course adjacent to Outfall 1 was also modelled as a storage basin as the existing culvert controls the upstream flow to some extent based on LiDAR topography data.
- Losses adopted in pre-developed conditions (Table 3-1) were used for existing conditions RORB model.
- The Kc routing parameter value was derived using the RORB regional equation #4. The addition of existing basins resulted in a changing the $d_{av}$ to 2.88. Since the routing parameter Kc was derived using Equation 4 in Table 3-2, a new Kc of 3.60 was adopted for existing conditions model.
- The existing conditions RORB model was simulated using the ARR2019 Intensity-Frequency-Duration (IFD) data and the selected Kc (routing parameter) values for the 1% AEP event across the ensemble of temporal patterns and a range of durations (from 10 min to 72-hour duration).

[5] Cardno TGM (June 2020), Jetty Rd Rezoning - Stage 2, Flood Study, Existing Conditions Report (Version 6)
[6] Drysdale Bypass Detailed Design – DP03 Drainage Design Report prepared by Jacobs for Decmil and MRPV, 2019

- The 1% AEP median peak flow at the external catchment flow inflow point at Jetty Road. The previous study estimated the existing peak flow at this location to be 1.1 $m^3/s$ (MRPV, 2019). The current model result indicated a peak flow rate of 0.81 $m^3/s$ at this location.

### 3.2.3 Development Conditions

- To create the post-development conditions RORB model, the new TIA fraction values were updated to account for the increase in imperviousness area associated with the proposed development.
- (TIA = 0.75), in line with Section 3.2.2. EIA and ICA are calculated from the same approach outlined in the existing conditions scenario.
- It was assumed that the seven main outfalls will cater for the Jetty Road Stage 2 development.
  - All sub-catchments not draining to identified outfalls under the existing conditions (i.e. either draining to the east or west but not through identified outfall locations) were diverted to a relevant outfall.
  - A summary of catchment area draining through each outfall under existing and post-development scenarios are summarised in Table 3-4.

TABLE 3-4 COMPARISON OF EXISTING AND POST-DEVELOPMENT CATCHMENT SIZE AND TIA

| Outfall | Existing Conditions | | Post-development Conditions | |
|---|---|---|---|---|
| | Area (ha) | TIA (%) | Area (ha) | TIA (%) |
| 1 | 124.7 | 33% | 142.5 | 74% |
| 2 | 267.1 | 21% | 273.5 | 46% |
| 3 | 14.6 | 5% | 14.6 | 75% |
| 4 | 17.8 | 5% | 17.8 | 75% |
| 5 | 6.9 | 5% | 8.3 | 75% |
| 6 | 5.2 | 5% | 11.8 | 75% |
| 7 | 17.1 | 5% | 27.2 | 75% |

- Reach types have also been modified to “piped” for all reaches within the development except for reaches along the proposed constructed waterway which were set as “excavated & unlined”.
- Additionally, runoff at Outfalls 3 and 4 were routed to the existing Council pipe network along Coriyule Road, which was considered the legal point of discharge (LPOD)
- The same Kc (routing parameter) value was adopted as the $d_{av}$ remains constant between the existing and developed conditions models.
- The developed conditions RORB model was run using the ARR2019 IFD data (ensemble of temporal patterns and a range of durations (from 10 min to 72-hour duration).
- The unmitigated post-development flows at Outfalls 1 – 7 are shown in Table 3-5.

TABLE 3-5 COMPARISON OF EXISTING AND POST-DEVELOPMENT (UNMITIGATED) 1% AEP MEDIAN FLOW AT OUTFALLS

| Outfall | Existing 1% AEP Median Flow ($m^3/s$) | Post-development Unmitigated 1% AEP Median Flow ($m^3/s$) |
|---|---|---|
| 1 | 1.83 | 8.67 |
| 2 | 5.56 | 6.51 |
| 3 | 0.78 | 2.80 |
| 4 | 0.95 | 3.66 |
| 5 | 0.51 | 1.34 |
| 6 | 0.40 | 1.28 |
| 7 | 0.30 | 1.26 |

- The post-development conditions model was then modified by incorporating proposed basins to estimate the required 1% AEP retardation storage and outfall configuration at each outfall.
- The catchment plan of the developed conditions RORB model is shown in Figure 3-6.

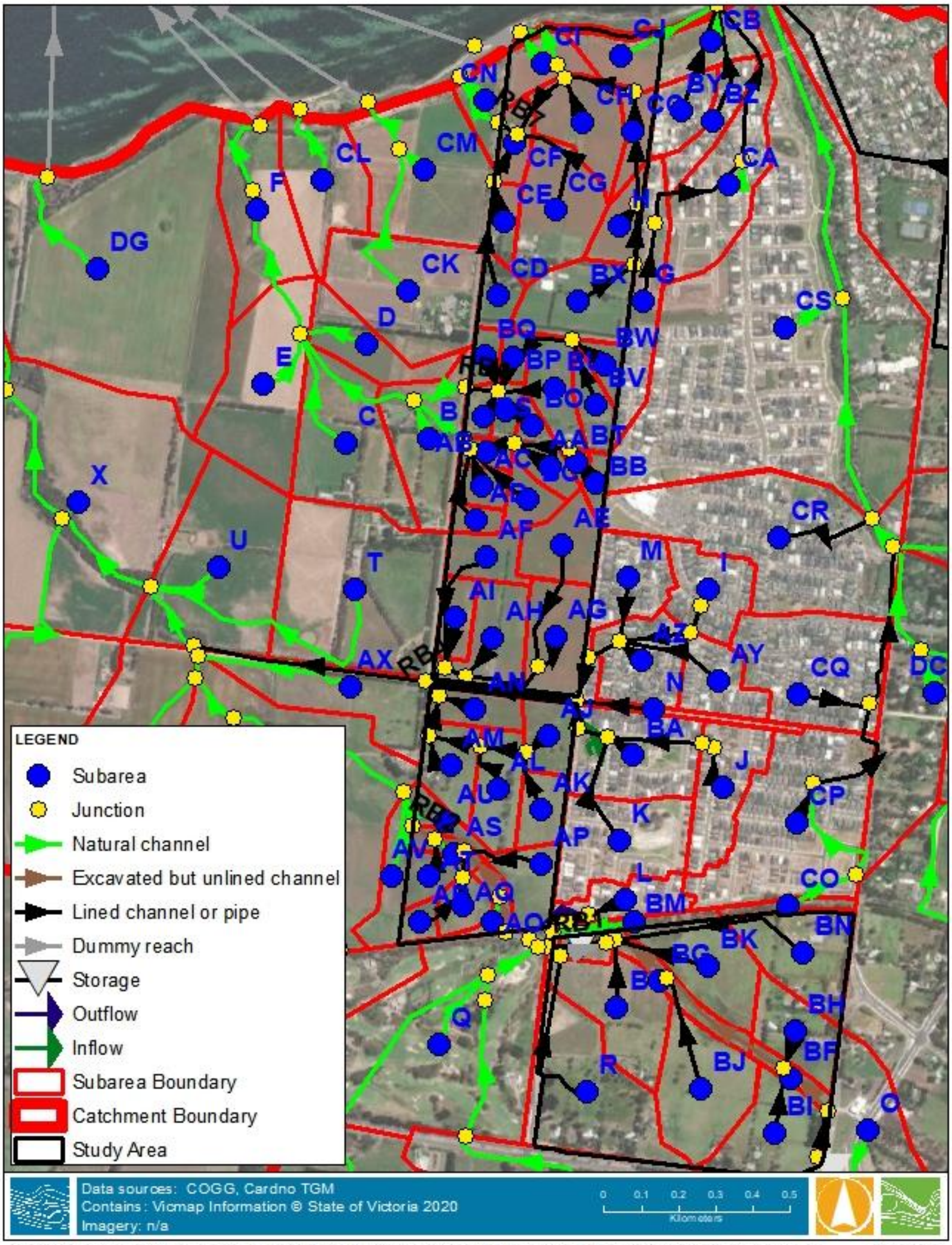


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**FIGURE 3-6 DEVELOPMENT CONDITION RORB MODEL CATCHMENT PLAN (ZOOMED INTO STUDY AREA)**

## 3.3 Retarding Basins (Outfalls 1 to 7 except Outfall 2)

The required 1% AEP retardation storage calculated from the post-development conditions RORB model at each outfall is summarised in Table 3-6. Modelling results indicated that the inclusion of RB1 at Outfall 1 resulted in 1% AEP peak flows at Outfall 2 being controlled to a rate below the existing 1% AEP flow. Therefore, no basin is proposed at Outfall 2. Further details of proposed no-detention strategy are described in Section 3.4.

Furthermore, it should be noted that the proposed basin strategy was based on the guiding principle of maintaining post-development peak flow rates at or below the pre-development levels and do not consider the additional requirement of maintaining post-development runoff volume at or below the pre-development levels. An additional volume assessment was conducted to satisfy the volume requirement using MUSIC modelling (refer to Section 4.3 for details) and the preferred option was to use ocean outfalls. Required changes in basin design to cater for the ocean outfall strategy are presented in Section 3.5.

TABLE 3-6 ESTIMATED 1% AEP FLOW RETARDATION STORAGE AT DRAINAGE OUTFALLS

| Outfall | Basin ID | Existing 1% AEP Median Flow ($m^3/s$) | Post-development Mitigated 1% AEP Median Flow ($m^3/s$) | Basin Outlet pipe size[a] | 1% AEP Retardation Storage ($m^3$)[b] | 1% AEP Retardation Storage Footprint Area ($m^2$)[c] |
|---|---|---|---|---|---|---|
| 1 | RB1 | 1.83 | 1.71 | 1 x 750 mm | 12,800 | 12,800 |
| 2 | - | 5.56 | 5.26 | - | - | - |
| 3 | RB3 | 0.78 | 0.56 | 1 x 600 mm | 2,790 | 4,413 |
| 4 | RB4 | 0.95 | 0.70 | 1 x 600 mm | 3,340 | 4,587 |
| 5 | RB5 | 0.51 | 0.37 | 1 x 450 mm | 1,410 | 2,440 |
| 6 | RB6 | 0.40 | 0.19 | 1 x 300 mm | 3,200 | 3,969 |
| 7 | RB7 | 0.30 | 0.25 | 1 x 300 mm | 10,300 | 7,950 |

Notes:
a. All outlet pipes were set at 1% gradient
b. Estimated using trapezoidal storage with 1 in 6 batters
c. Taken as the top surface area with 300 mm freeboard. Exclude additional footprint required for access track etc.

It should be noted that the above basin outlets were only sized to control 1% AEP flows to the existing conditions. These outlets should be configured to mitigate smaller magnitude events (e.g. 50% AEP) back to existing conditions flow rates during later design stages.

## 3.4 No Detention Strategy at Outfall 2

Clause 56.07-04 of the of the Victorian Planning Provisions typically requires that a stormwater management system be designed to ensure that peak flows leaving the subdivision are restricted to pre-development flows, unless increased flows are approved by the relevant drainage authority and there are no detrimental downstream impacts. A no-detention strategy is on occasion warranted, typically either because of reduced flood risk further downstream or because the unattenuated flows makes a negligible change to peak flows.

The RORB model identifies the unattenuated 1% AEP peak flow rate at Outfall 2 is below the existing conditions flow rate due to retardation asset proposed at Outfall 1 which alters the timing of flows leading to Outfall 2. Therefore a no detention strategy is proposed at Outfall 2

**Notional risks** associated with a no-detention strategy include:

- Intensify the harmful impacts of flooding on adjacent properties;
- Impact on values of downstream sensitive environments/habitats;
- Impact on the capacity of existing infrastructure to convey flood flows;
- Impact erosion processes in the receiving waterways and land; and
- Fundamentally change, affect or limit the future development potential of the downstream properties.

Table 3-7 summarises how the above risk may apply to the site.

**TABLE 3-7 NO DETENTION RISKS AND DESIGN RESPONSES (IF APPLICABLE)**

| Risks | Comment |
|---|---|
| Intensify the harmful impacts of flooding on adjacent properties | The change in flows as a result of unattenuated flow (i.e., the no-detention strategy) is considered negligible (lower than the existing conditions), as demonstrated by the hydrological modelling. |
| Impact on values of downstream sensitive environments/habitats | Provided that there is no polluted and/or sediment laden runoff discharged from the site, the proposed no detention strategy will not impact on downstream riparian values. This will be achieved with the provision of water quality assets, such gross pollutant traps and raingardens and should include suitable controls during the construction stages. |
| Impact on the capacity of existing infrastructure to convey flood flows | Any changes in flood depths are likely to be negligible and contained within the waterway channel as the unattenuated peak flow is slightly lower than the existing conditions. |
| Impact on erosion processes in the receiving waterways and land | Any relative changes to erosion processes can be mitigated with the inclusion of appropriate erosion protection measures. |
| Change, affect or limit the future development potential of the downstream properties | The change in flows as a result of unattenuated flow rates is not considered to impact on the development potential of the downstream properties. These are predominantly located within current Farming Zones. Additionally, flows will likely remain contained within the waterway channel.<br><br>However, it is understood that the post-development runoff volumes are likely to be exceeding existing volumes at each outfall. Additional strategies presented in Section 4.3 will be required to manage post-development runoff volumes at outfalls. |

## 3.5 Preferred Volume Management Strategy – Ocean Outfalls

An alternative drainage strategy involving diversion of stormwater runoff flows to an ocean outfall to deal with increased runoff volumes associated with the proposed development has been investigated. This option is required to maintain the post-development runoff volumes at all outfalls and to minimise adverse impacts on the downstream properties typically associated with increased frequency of flows and changed hydrological conditions.

- Outfalls 3 and 4 drain to the existing council pipe network with an ocean outfall. Volume management requirement is not applicable to these two outfalls.
- Similarly, due to close proximity of Outfall 7 to Port Phillip Bay, the impact on downstream properties are minimal.
- Outfalls 2, 5 and 6 discharge to existing waterways west of the site through private property. Without additional measures such as stormwater harvesting and infiltration within the development, the post-development runoff volume will exceed the pre-development volume (Section 4.3).
- In response to this, an alternative strategy would be to divert the excess flows from these basin outfalls to basins where volume reduction is not critical. The following options were investigated:
  - Outfall 2 diverts all excess flows to RB 3
  - Outfall 5 diverts all excess flows to RB 4
  - Outfall 6 diverts all excess flows to RB 7
- The diversion arrangement needs to match not only the total annual runoff volume but also the baseflow volumes under existing conditions to replicate the natural hydrological cycle. A potential arrangement involving connection of two basins via pipes with sufficient capacity and inlets placed above the invert of upstream basin to allow low flow to pass through.
- MUSIC modelling was used to develop an indicative diversion arrangement in the form of low flow bypass and high flow bypass (Section 4.3). Resulting bypass rates and indicative connection pipe sizes are summarised in Table 3-8. These flow rates are to be further optimised during later design of drainage assets.

**TABLE 3-8 INDICATIVE DIVERSION ARRANGMENTS**

| Outfall | Low Flow Bypass ($m^3/s$) | High Flow Bypass ($m^3/s$) | Indicative Connection Pipe* |
|---|---|---|---|
| 2 | 0.006 | 0.04 | 225 mm at 1% slope |
| 5 | 0.0008 | 0.15 | 375 mm at 1% slope |
| 6 | 0.00015 | 0.25 | 450 mm at 1% slope |

* These pipe sizes and slopes are to be finalised during concept design taking into consideration basin levels and blockage risks of smaller pipes may necessitate the need for an orifice plate.

- As basins at outfalls 3, 4 and 7 are receiving additional catchment flows under this scenario, the basin storage volumes need to be increased. A summary of total retarding volume at these basin locations under this volume management option are summarised in Table 3-9.

**TABLE 3-9 ESTIMATED 1% AEP FLOW RETARDATION STORAGE AT DRAINAGE OUTFALLS RECEIVING ADDITIONAL FLOWS**

| Outfall | Basin ID | Existing 1% AEP Median Flow ($m^3/s$) | Post-development Mitigated 1% AEP Median Flow ($m^3/s$) | 1% AEP Retardation Storage ($m^3$)[a] | 1% AEP Retardation Storage Footprint Area ($m^2$)[b] |
|---|---|---|---|---|---|
| 3 | RB 3 | 0.78 | 0.58 | 2,880 | 4,413 |
| 4 | RB 4 | 0.95 | 0.58 | 4,850 | 6,914 |
| 7 | RB 7 | 0.30 | 0.24 | 17,300 | 13,356 |

Notes:

a. Estimated using trapezoidal storage with 1 in 6 batters

b. Taken as the top surface area with 300 mm freeboard. Exclude additional footprint required for access track etc.

## 3.6 Coriyule Road Pipe Capacity Assessment

In addition to retarding post-development flows to existing flow rates, the 1% AEP peak flow at the LPOD1 (Figure 3-1) was compared with the pipe capacity of the existing Council pipe along Coriyule Road where drainage Outfalls 3 and 4 are assumed to be discharged (to Pit No. 9 and 10 respectively)[7].

A preliminary assessment conducted using the manning's equation as part of the draft SWMP was replaced with a detailed hydraulic assessment conducted using 1D/2D hydraulic model was developed using TUFLOW. The pipes were incorporated into the model based on the design plans[8] provided by Council and the terrain DEM utilised LiDAR data. A summary of modelling approach and findings are presented in sections below.

### 3.6.1 Model simulation

The hydraulic model was simulated for the 1% AEP event for the 1.5 and 9-hour storm events for both the existing and developed conditions. The results were then spliced to produce the maximum depth for existing conditions. The maximum depth results (Figure 3-7) show that 1% AEP flood depths are generally less than 300mm through the site, with the exception of the location of the two existing dams. The results also indicate that flow path along the waterway (location of former outlet) is relatively confined due to the slope of the topography of the site.

### 3.6.2 Pipe Inflows

The RORB model developed as part of the Stage 2 SWMS was used to extract flows at three locations for use in the hydraulic model. The Jetty Road Stage 1 inflows were split between the 1050mm diameter and 750mm diameter culverts located upstream of the Outfall 3 & 4 inflows. This split representing the outflows from the two major basins discharging from Stage 1.

[7] Issued for Construction Drawings of "Proposed Drainage Improvement Coriyule Road, Curlewis (DWG No 201431D01-D09). Dated 25/01/2016

[8] Council drawings for the extension work (DWG No. 2017074D01-D13 dated 21/06/2021) and an internal technical memo (dated 01/08/2018) with preliminary design calculations were received from Council and reviewed as part of the current work. The technical memo suggests the proposed extension work is to convey all the flows from existing DN 1200 or 3/DN 750 pipes. Design drawings show a significant change in pipe slope from the end of previous pipeline (1% to 0.1%) which reduce the pipe capacity. It is understood the initial design concept was modified at a later date by external consultants. It was later confirmed by Council the design drawings provided as the as-constructed drawings. Therefore, pit and pipe data were used as it is in the analysis.

The existing conditions assumes no other flows enter the pipe network beyond the Stage 1 basin outfalls. Existing runoff from the outfall 3 & 4 locations are assumed to be dispersed and to flow along the natural drainage paths.

The developed conditions incorporate the outfall flows from the RORB modelling undertaken as part of the Stage 2 Rezoning SWMS.

A summary of the peak flows model inputs for the existing and developed conditions is shown in Table 3-10.

**TABLE 3-10 1% AEP FLOWS WITHIN THE COUNCIL PIPE ALONG THE CORIYULE ROAD (RORB)**

| Council Pipe Location | 1% AEP Median Flow Rate ($m^3/s$) | |
|---|---|---|
| | Existing | Developed |
| **Outfall 3 Connection** | N/A | 0.58 (1.5 Hour) |
| **Outfall 4 Connection** | N/A | 0.50 (1.5 Hour)<br>0.58 (9-hour) |
| **Outfall to Coriyule Rd Main Drain (Jetty Rd Stage 1)** | 2.01(1.5 Hour) | 2.01 (1.5 Hour) |

### 3.6.3 Model Results

The modelling results shown in Figure 3-7 - Figure 3-9 identify the increase in hydraulic grade line in a 1% AEP event within the Coriyule Road pipe. The increase in HGL occurs as the additional inflows from RB 3 and RB4 enter the pipe at chainages 282 m and 362m. As the flows increase, the HGL begins to exceed the surface elevation at around chainage 600m.

The existing conditions also show the system under pressure, the HGL begins to exceed the surface at around chainage 900m (at the change in pipe slope) and continues above the surface through the low spot at the former outlet into the property located on the northern corner of Coriyule and Scarborough Road (~Chainage 1000m). Through the low lying area, the Coriyule Road pipe is converted from a single 1200mm diameter pipe to a set of 3x 750mm diameter pipes.

As the pipe continues, the hydraulic grade line in the developed conditions returns to similar (slightly higher) levels compared to the existing conditions before discharging to the bay.

Peak flows within the pipe downstream of Outfalls 3 & 4 generally increase from 1.91 $m^3/s$ to 2.45 $m^3/s$. Velocities within the pipes are also increased following development. The single 1200mm diameter section upstream of the Stage 2 extension increases around 0.1 m/s (2.0 m/s to 2.1 m/s). Through the three 750mm diameter section, peak velocities increase from 0.40 m/s (1.44 to 1.85 m/s). Downstream of the transition back to a single 1200mm diameter, velocities generally increased by around 0.30 – 0.50 m/s.

#### Open Pits

A secondary modelling scenario was assessed where all pits were allowed to surcharge (connected to the 2d surface). This surcharging scenario showed a peak surcharge of 0.33$m^3/s$ which flows into the existing dam at property located on the northern corner of Coriyule Rd and Scarborough Rd under existing conditions and 0.73 $m^3/s$ in the developed conditions scenario. It is estimated that this is likely lower than 1% AEP peak flows that entered the site prior to the construction of the Stage 2 Coriyule Road pipe extension.

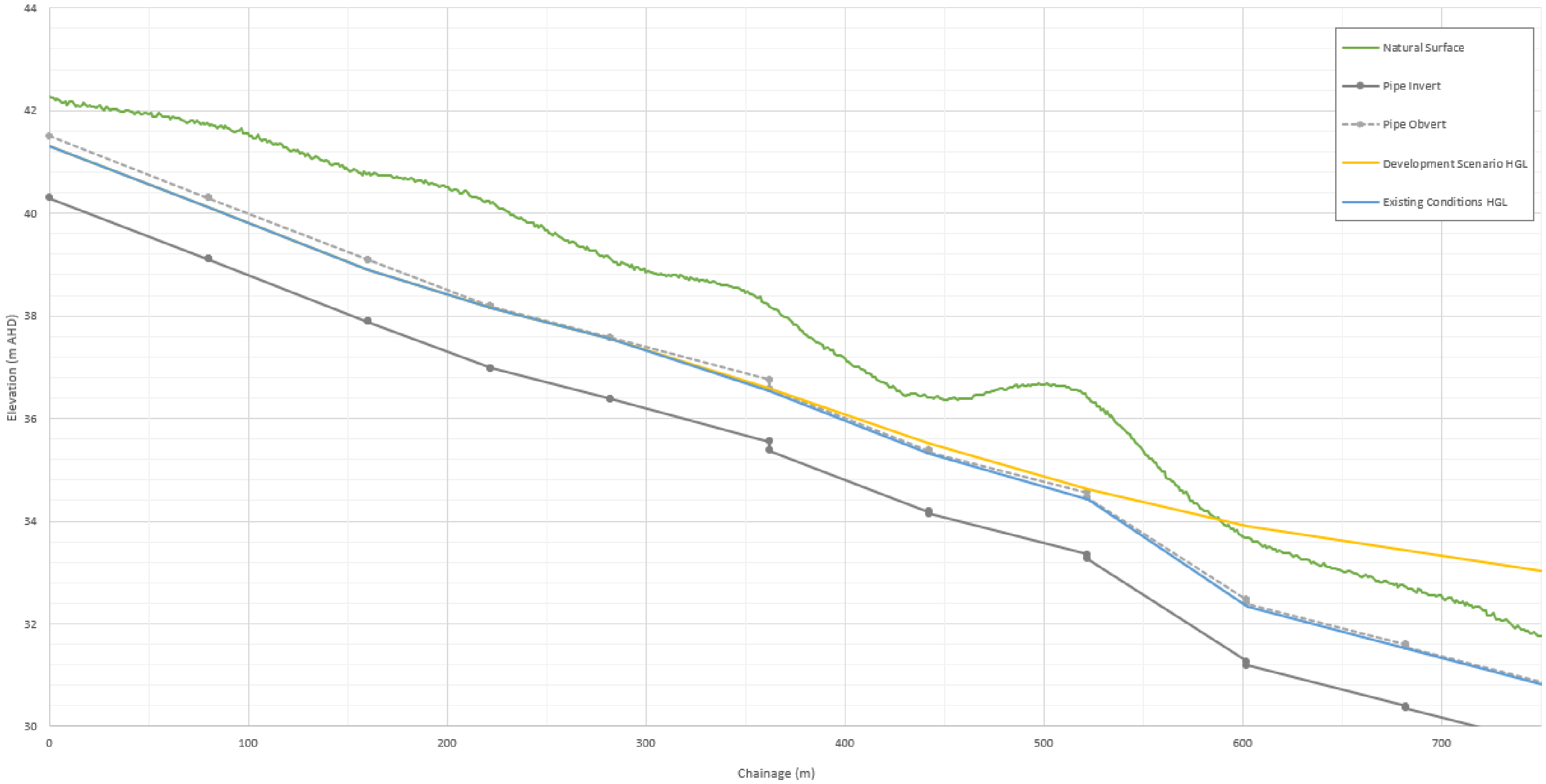


**FIGURE 3-7 HGL ANALYSIS – CH 0 – 750M**

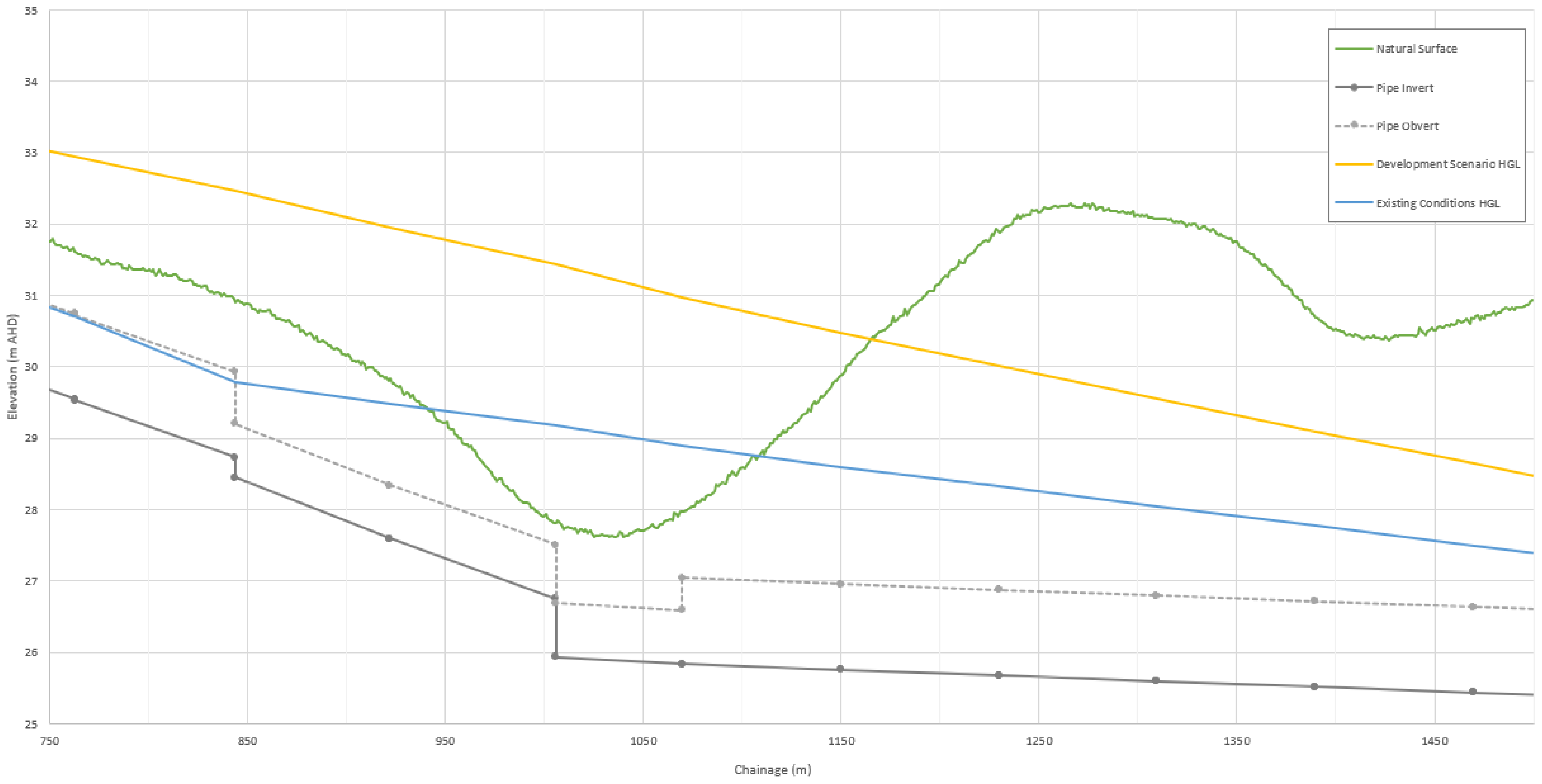


**FIGURE 3-8 HGL ANALYSIS – CH 750 – 1500M**

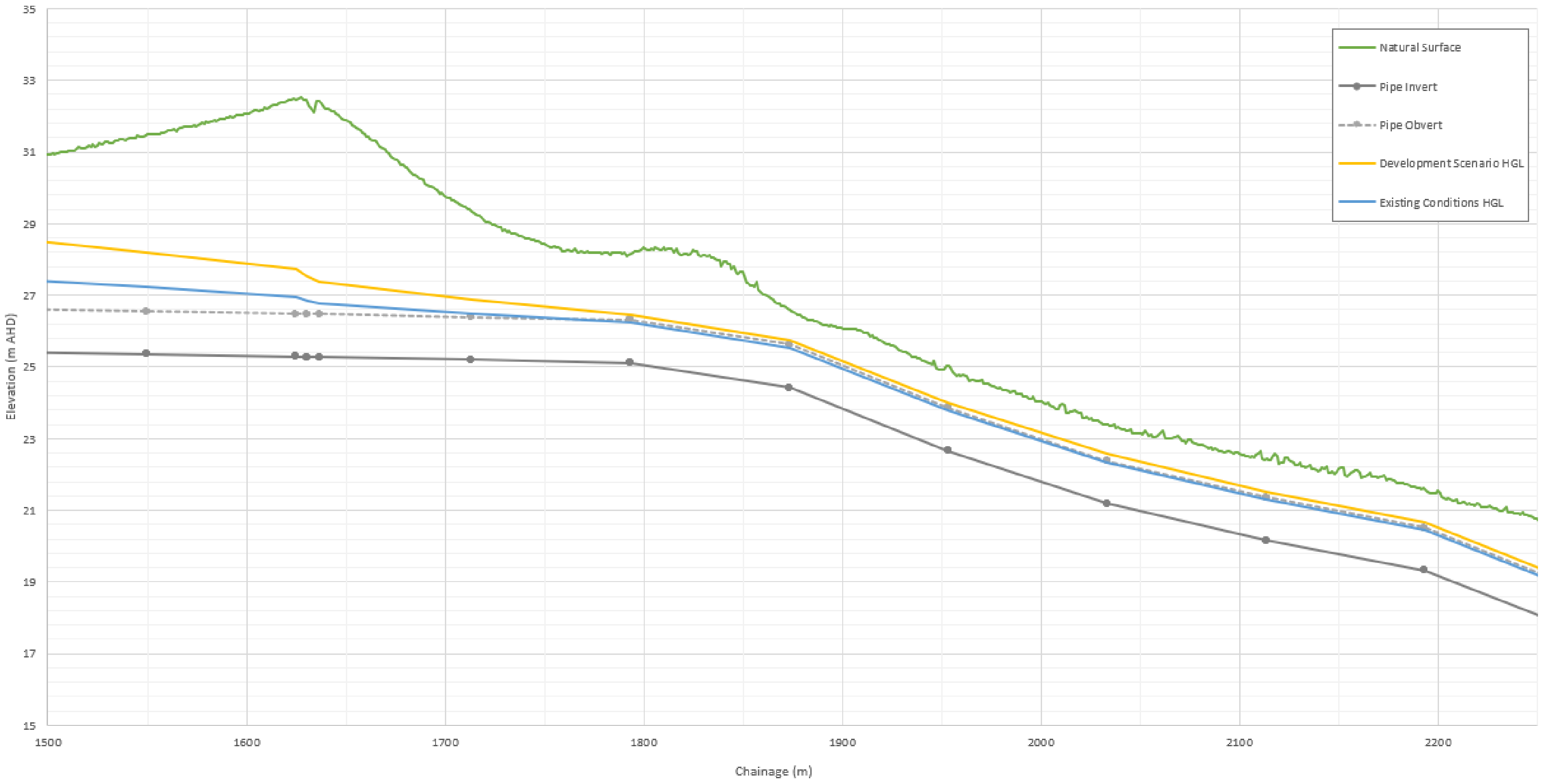


**FIGURE 3-9 HGL ANALYSIS – CH 1500 – 2250M**

### 3.6.4 Outcome of the Assessment

Hydraulic modelling has been undertaken to assess the impact of the connection of flows from the Jetty Road Stage development on the HGL of the Coriyule Road pipe.

The modelling shows that the pipeline in its current format has a HGL that is above the surface elevation in a 1% AEP event. The inclusion of the additional inflows as part of the Stage 2 proposed development will increase the HGL in the sealed network as the peak flows within the pipe downstream of Outfall 3 & 4 increase from 1.91 $m^3/s$ to 2.45 $m^3/s$.

Should flows from the Outfall 3 & 4 basin not be able to flow into the pipe in a 1% AEP event, it is likely that spilling of the basin would occur. The design of the basin should ensure that spilling of the basins resulting in overland flow onto Coriyule Road should be done so it can occur in a manner that ensures flood hazard is considered.

If required, options to reduce the increase on HGL may be further assessed including:

- Reduced peak outflow rates which would result in an increase in Basin size.
- Diversion of additional area within catchment to the secondary Ocean outfall (further north)
- Spilling of the pipe network in large storm events into the existing waterway which crosses Coriyule Road (property on northern corner of Coriyule and Scarborough Rd).

## 3.7 Constructed Waterway

Two constructed waterways are proposed upstream of Outfall 1 and 2 generally following the existing waterway alignment. These waterway reaches are proposed to be designed as a compound waterway incorporating a low flow channel (LFC) within a high flow channel (HFC) shown in Figure 3-10. This has been designed in accordance with Melbourne Water's Waterway Corridors guidelines (Melbourne Water 2019). The LFC is designed to convey up to 1 EY flows and the HFC to convey 1 % AEP flows in post-development conditions. Summary of key waterway design flows is presented in Table 3-11.

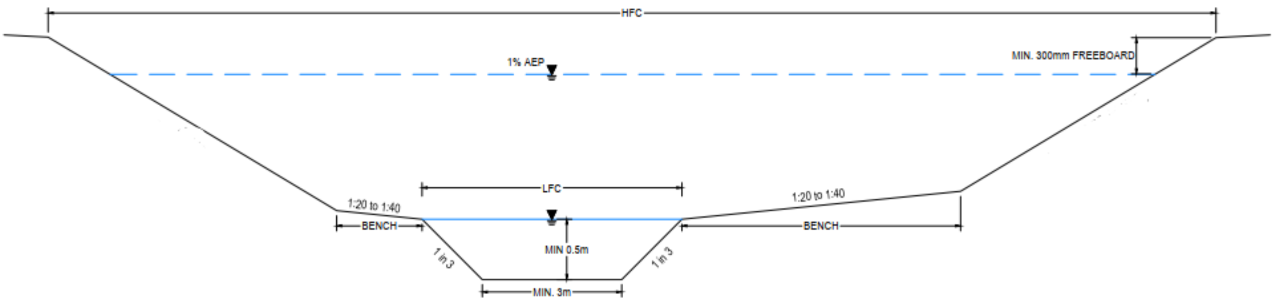


**FIGURE 3-10 TYPICAL COMPOUND WATERWAY CROSS-SECTION (SOURCE: MW CONSTRUCTED WATERWAY STANDARD DRAWING)**



**TABLE 3-11 WATERWAY DESIGN FLOWS**

| Location | LFC Design Flow Rate (1 EY) ($m^3/s$) | HFC Design Flow Rate (1% AEP) ($m^3/s$) |
|---|---|---|
| Upstream of Outfall 1 | 1.05 | 6.39 |
| Upstream of Outfall 2 | 0.81 | 5.29 |

It should be noted that peak flow rates shown in Table 3-11 were extracted at the downstream end of each waterway reach. Hence these design flows represent the maximum flow rate to be conveyed within that reach. This capacity is not required through the entire reach length and overall waterway width may reduced in upstream sections of each reach width once inflow connections are confirmed at a later design stage. An indicative waterway cross section that complies with the Constructed Waterway Guidelines (MW, 2019) is shown in Figure 3-11.

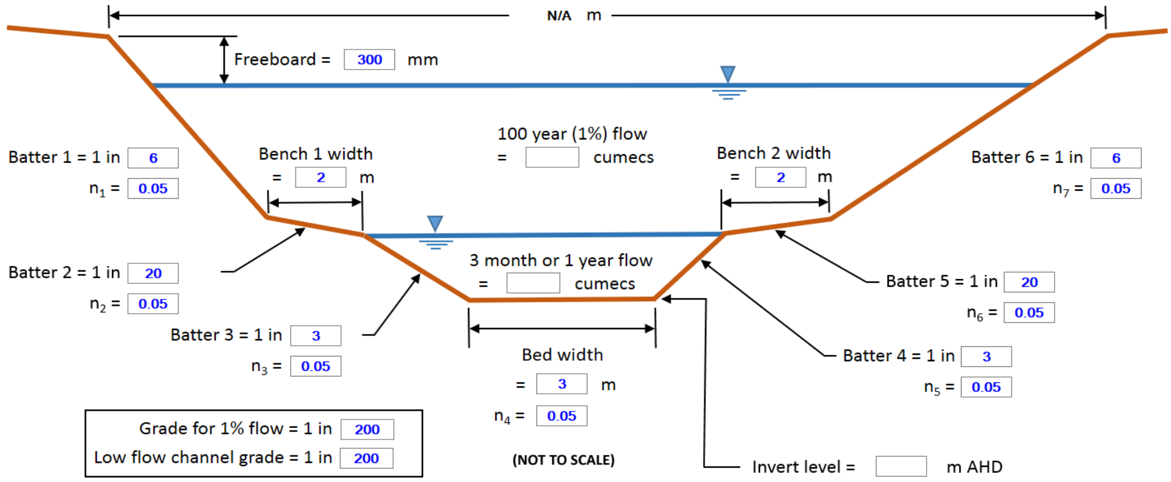


**FIGURE 3-11 PROPOSED WATERWAY CROSS-SECTION**

- The existing waterways draining to Outfall 1 and 2 have an average longitudinal slope of 1.5% to 2%. The average design longitudinal slope was assumed to be 1 in 200 across both waterway corridors to allow for pool-riffle and pool-run sequences.
- The performance of the waterway cross-sections were assessed using PC-Convey (Appendix C).
- Water surface elevation of the two reaches for low flow (1 EY) and high flow (1% AEP) in post-development conditions are shown in Figure 3-12 to Figure 3-13.
- The waterway corridor width for constructed waterways is defined as a factor of the hydraulic width of the main channel of the waterway. Hydraulic width is defined as the width of the water surface in metres at the 1% AEP post development flow level in the channel (Melbourne Water 2013)[9].
  - A typical constructed waterway corridor showing the extent of hydraulic width and overall corridor width is shown in Figure 3-14.
  - Resulting hydraulic width and corresponding minimum waterway corridor width as per Melbourne Water Waterway Corridor Guidelines (Melbourne Water 2013) are summarised in Table 3-12.

[9] Melbourne Water, 2013, Waterway Corridors: guidelines for greenfield development areas within the Port Phillip and Westernport region

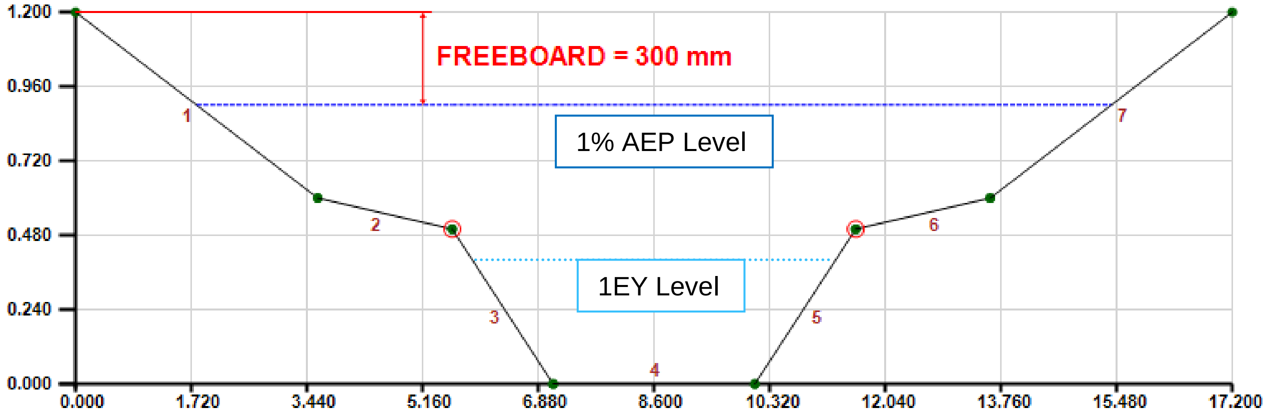


FIGURE 3-12 LFC & HFC PC CONVEY ANALYSIS (OUTFALL 1)

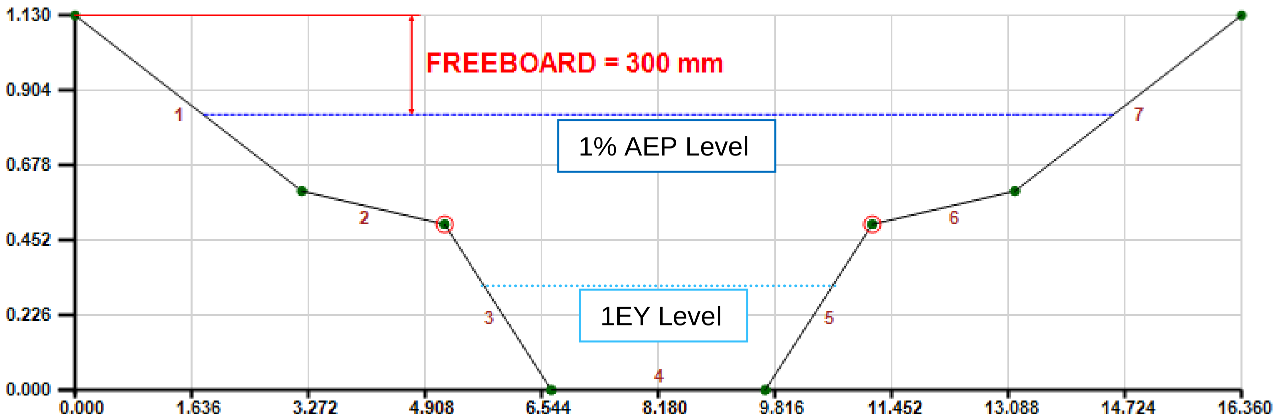


FIGURE 3-13 LFC & HFC PC CONVEY ANALYSIS (OUTFALL 2)

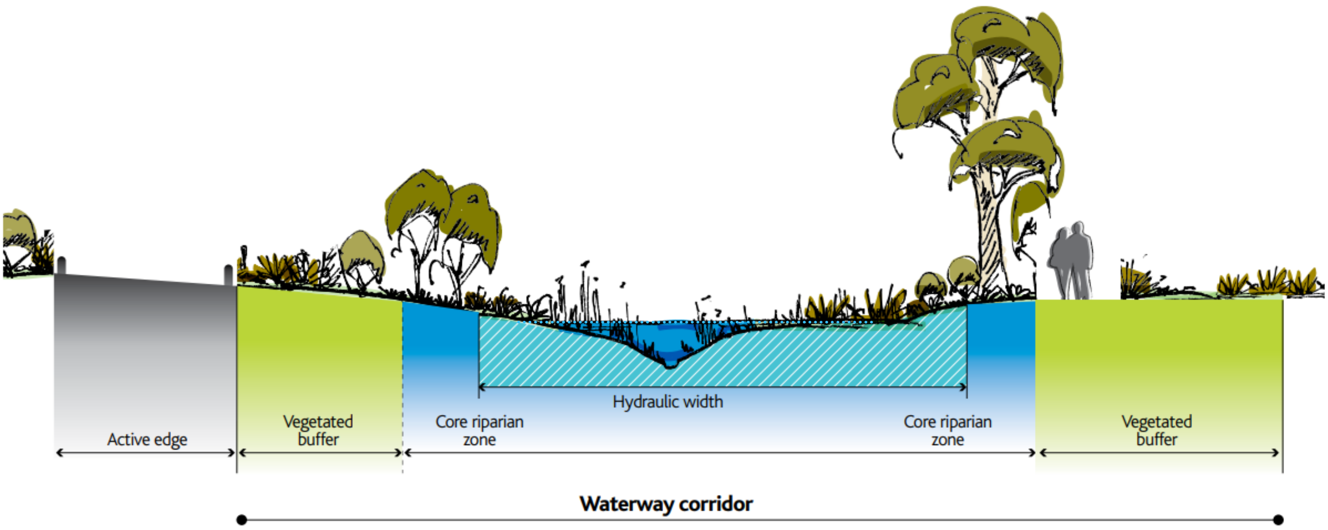


FIGURE 3-14 EXAMPLE OF SETBACK SUBZONES FOR CONSTRUCTED WATERWAYS (MELBOURNE WATER, 2013)

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**TABLE 3-12 MINIMUM WATERWAY CORRIDOR WIDTH**

| Location | Upstream of Outfall 1 | Upstream of Outfall 2 |
|---|---|---|
| Hydraulic width (1% AEP flow width) | 13.6 m | 12.36 m |
| HFC extent (with 300 mm freeboard) | 17.2 m | 16.36 m |
| Minimum corridor width assuming there are roads on both sides of the corridor to allow vehicular access | 40 m | 40 m |
| Minimum corridor width, if a shared trail/maintenance track to be added to either side of channel (within vegetated buffer) | 45 m | 45 m |

# 4 WATER QUALITY MANAGEMENT

The following section of the SWMS details the Water Sensitive Urban Design (WSUD) assets proposed to treat runoff from the development. The water quality treatment targets established by the Urban Stormwater Best Practice Guidelines (CSIRO, 1999) should be achieved as a minimum to protect ecological values within Port Philip Bay. The load reduction targets for key pollutants are as follows:

- 80% of total suspended sediments (TSS);
- 45% of total nitrogen (TN);
- 45% total phosphorous (TP); and,
- 70% gross pollutants.

Additionally, the post development runoff volume should not exceed the pre-development runoff volume as per the Jetty Road Urban Growth Plan[1].

The WSUD strategy (Table 4-1) was informed by the current best practice industry methods, City of Greater Geelong MUSIC guidelines[10], and Melbourne Water MUSIC guidelines[11]. In addition to typical stormwater treatment technologies, lot and precinct scale stormwater harvesting and infiltration opportunities to meet the runoff volume management targets were investigated. It should be noted that the current WSUD strategy does not include potential street-scape measures such as passively irrigated street trees. Feasibility of implementing these measures should be considered in later design stages.

**TABLE 4-1 WSUD ASSET SELECTION CRITERIA**

| Treatment Type | Catchment threshold | Recommended asset for smaller catchment (< threshold area) | Recommended asset for larger catchment (> threshold area) |
|---|---|---|---|
| Primary | 5 ha | Gross pollutant trap | Sediment pond |
| Secondary/tertiary | 10 ha | Bioretention | Wetland |

MUSIC modelling (Version 6.3) was undertaken to estimate the WSUD asset sizing and investigate runoff volume reduction opportunities. Geelong North 20 year (1971 – 1990) 6-minute MUSIC climate template (available from the City of Greater Geelong MUSIC modelling guidelines) was adopted for the stormwater quality modelling. The MUSIC model schematic is shown in Figure 4-1.

[10] City of Greater Geelong MUSIC Modelling Guidelines. Available at https://www.geelongaustralia.com.au/idm/documents/item/8cf4f273fe1120f.aspx
[11] Melbourne Water MUSIC modelling guidelines (2018). https://www.melbournewater.com.au/sites/default/files/2018-03/Music-tool-guidelines.pdf
Melbourne Water Constructed Wetland Manual https://www.melbournewater.com.au/planning-and-building/developer-guides-and-resources/standards-and-specifications/constructed-0
Melbourne Water Biofiltration systems in Development Services Schemes guidelines https://www.melbournewater.com.au/media/14586/download

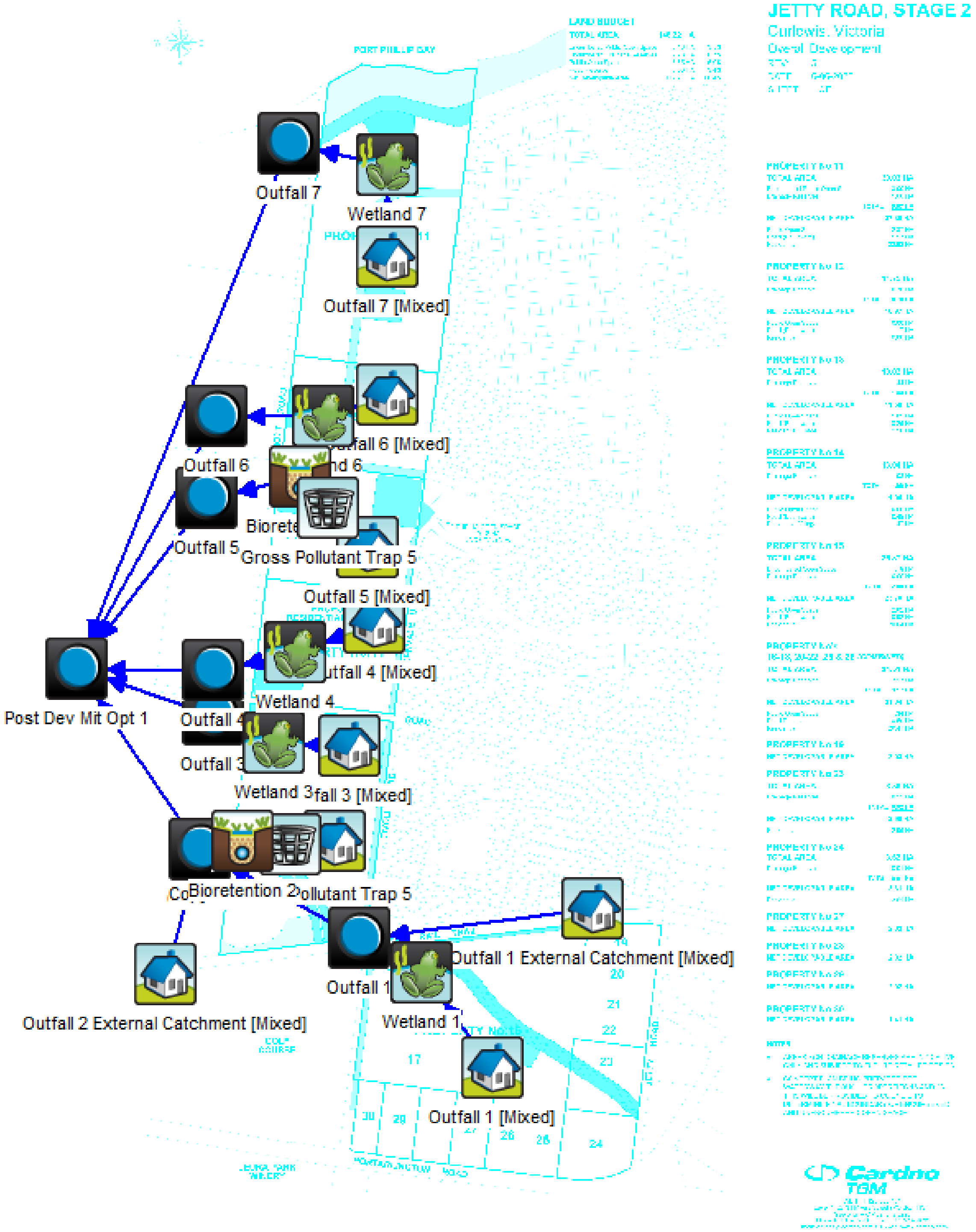



**FIGURE 4-1 MUSIC MODEL SCHEMATIC**

## 4.1 Water Quality Catchments

Since no hydrologic routing was applied in MUSIC modelling, water quality catchments were represented using a simplified/lumped approach. Catchment FI values were set as per Melbourne Water MUSIC guidelines (i.e., taken as TIA of RORB model subareas). A catchment summary is presented in Table 4-2.

**TABLE 4-2 JETTY ROAD STAGE 2 DEVELOPMENT AREAS TO BE TREAT AT EACH OUTFALL**

| Catchment | Existing | | Post-development | |
|---|---|---|---|---|
| | Area (ha) | FI (%) | Area (ha) | FI (%) |
| Outfall 1 | 46.5 | 20 | 52.9 | 67(1) |
| Outfall 2 | 11.3 | 5 | 11.3 | 60(1) |
| Outfall 3 | 14.6 | 5 | 14.6 | 75 |
| Outfall 4 | 17.8 | 5 | 17.8 | 75 |
| Outfall 5 | 6.9 | 5 | 8.3 | 75 |
| Outfall 6 | 5.2 | 5 | 11.8 | 75 |
| Outfall 7 | 17.1 | 5 | 27.2 | 75 |

(1) The typical FI for a standard residential development is 75%. A lower FI was used in MUSIC after taking into account waterway corridor and drainage reserves, etc.

In addition to the Jetty Road Stage 2 catchments listed in Table 4-2 above, an additional 89.6 ha (FI - 39%) external catchment drains through outfall 1. Similarly, another 119.7 ha (FI – 10%) drains through Outfall 2.

Pervious area soil parameters were adopted as follows:

- Soil Store Capacity = 120 millimetres
- Field Capacity = 50 millimetres

## 4.2 Water Quality Assets

For the purposes of this assessment, options which consider both a conventional constructed wetland treatment system and bioretention system have been investigated. Constructed wetlands are the City's preferred stormwater treatment asset type, and being the larger of the two shall be used for the land yield assessment.

Whilst it is understood that the City of Greater Geelong have a preference that treatment assets remain as wetlands, at the planning permit stage bioretention systems may be considered if the following is addressed to the satisfaction of Council:

- Peer Review detailed design (suitably qualified specialist)
- Construction, maintenance and monitoring Plan
- Construction supervision (media installation, in particular by a suitably qualified specialist with council officer involvement)
- Self-reporting monitoring and handover compliance requirements
- Agreed handover checklist and or altered defects liability period

WSUD asset sizes were estimated to achieve the BPEM target at each outfall for runoff generated from Jetty Road Stage 2 development excluding any external catchment flows draining to outfalls. A summary of key WSUD asset parameters is listed in Table 4-3. High flow bypass was set to 1 EY flow (extracted from RORB model). The recommended WSUD asset types and their sizing at each of the main outfalls are listed in

Table 4-4 while the corresponding overall treatment train effectiveness is summarised in Table 4-5.

An estimate of total asset footprint required were also provided (Table 4-4) by increasing the asset's treatment area by two times to allow for additional space required for freeboard, access tracks etc (based on existing development areas). These footprint area estimates are preliminary only and to be confirmed during concept and functional design stages. It was also assumed that the water quality assets will be co-located with the retarding basin where the water quality asset such as wetland will be at the base of the retarding basin.

**TABLE 4-3 WATER QUALITY ASSET PARAMETER SUMMARY**

| Asset Type | Key Modelling Parameters |
|---|---|
| GPT | ■ Modelled using generic node<br>■ Only applied 70% reduction in gross pollutants |
| Sediment pond | ■ Sizing was determined using Fair and Geyer Eq. to capture 95% of coarse particles ≥125µm diameter from the peak 1 EY flow and 5 year clean out frequency (Refer to Appendix D for sediment pond sizing)<br>■ Modelled permanent pool volume (PPV) was equal to the sediment pond volume from halfway up the sediment accumulation zone to the normal water level (NWL)<br>■ When sediment pond is proposed upstream of a wetland, it is included within the wetland node as "inlet pond volume" |
| Bioretention | ■ All bioretention systems were modelled with a submerged zone as per City's guidelines<br>■ Filter media depth = 500 mm<br>■ Submerged zone depth = 450 mm<br>■ Hydraulic conductivity = 100 mm/hr |
| Wetland | ■ Sediment pond volume is included as the inlet pond volume<br>■ Extended detention depth = 0.35 m<br>■ PPV = 0.4 x surface area<br>■ Detention time ~ 72 hr |

TABLE 4-4 RECOMMENDED WATER QUALITY ASSET AND THEIR SIZES AT MAIN OUTFALLS

| Outfall | Treatment Area | | | | | Total Asset Footprint incl provisions for access and maintenance (m²) a |
|---|---|---|---|---|---|---|
| | GPT | Sed Pond surface area (m²) | Biofilter surface area (m²) | Wetland surface area (m²) | Total asset treatment area (m²) | |
| 1 | | 1,100 | | 12,000 | 13,100 | 26,200 (b) |
| 2 – Opt 1(c) | | 400 | | 2,100 | 2,500 | 5,500 |
| 2 – Opt 2(d) | yes | | 900 | | 900 | 1,980 |
| 3 | | 450 | | 3,700 | 4,150 | 9,130 |
| 4 (e) | | 600 | | 4,500 | 5,100 | 11,220 |
| 5 – Opt 1(c,e) | | 350 | | 2,500 | 2,850 | 6,270 |
| 5 – Opt 2(d,e) | yes | | 700 | | 700 | 1,540 |
| 6 | | 400 | | 3,000 | 3,400 | 7,480 |
| 7 | | 700 | | 7,200 | 7,900 | 17,380 |

Notes:

a. Total asset footprint including provisions to access tracks, batter slopes etc. was taken as 2.2 times the asset's treatment area as agreed with City of Greater Geelong except for Outfall 1 (Refer to note b for further details). This is only a preliminary estimate based on best judgement and previous examples as an indication of the likely total footprint. Asset footprint will be governed by site specific conditions such as final design surface and slope, reserve shape and orientation of assets etc. Therefore, the actual required may vary from the preliminary estimate. The required asset footprint is to be confirmed at concept and functional design stages.

b. The drainage asset design for Outfall 1 has been progressed to functional design at the time of reporting. The current functional design layout shows the required WSUD asset can be implemented within a reserve footprint equal to two times the treatment area. Therefore, the asset footprint was taken as two times the treatment area for Outfall 1. The functional design is currently being reviewed by the City of Greater Geelong. The final WSUD asset footprint at Outfall 1 is to be confirmed within a separate stormwater management strategy for Outfall 1, which is being prepared concurrently.

c. Option 1 represents the WSUD treatment train including a constructed wetland and a sediment pond. This option to be used for land yield assessment. Option 2 which consists of a bioretention system could be considered in future planning permit and design phases in discussion with Council.

d. Used for pollutant load reduction and volume assessment calculations.

e. WSUD assets at Outfall 4 and 5 are considered to be privately owned at the time of reporting

**TABLE 4-5 TREATMENT TRAIN EFFECTIVENESS SUMMARY (EXCLUDING EXTERNAL CATCHMENTS DRAINING THROUGH OUTFALL 1 AND 2)**

| Outfall | Pollutant | Post-development Inflow Load (kg/year) | Post-development Outflow Load (kg/year) | Load Reduction | BPEM Target Met |
|---|---|---|---|---|---|
| 1 | TSS | 31,700 | 6,400 | 80% | Yes |
| | TP | 64.9 | 19.7 | 70% | Yes |
| | TN | 457 | 219 | 52% | Yes |
| | Gross pollutants (GP) | 7,150 | 336 | 95% | Yes |
| 2[a] | TSS | 6,160 | 447 | 93% | Yes |
| | TP | 12.6 | 6.91 | 45% | Yes |
| | TN | 88.8 | 26.8 | 70% | Yes |
| | GP | 1,410 | 11.6 | 99% | Yes |
| 3 | TSS | 9,770 | 2,030 | 79% | Yes |
| | TP | 19.8 | 6.01 | 70% | Yes |
| | TN | 139 | 67.4 | 52% | Yes |
| | GP | 2,130 | 68.6 | 97% | Yes |
| 4 | TSS | 12,000 | 2,430 | 80% | Yes |
| | TP | 24.5 | 7.25 | 70% | Yes |
| | TN | 171 | 82 | 52% | Yes |
| | GP | 2,600 | 73.5 | 97% | Yes |
| 5 | TSS | 5,550 | 452 | 92% | Yes |
| | TP | 11.3 | 6.23 | 45% | Yes |
| | TN | 79.1 | 26.2 | 67% | Yes |
| | GP | 1,210 | 16 | 99% | Yes |
| 6 | TSS | 7,880 | 1,600 | 80% | Yes |
| | TP | 16.1 | 4.84 | 70% | Yes |
| | TN | 113 | 54.4 | 52% | Yes |
| | GP | 1,720 | 42.1 | 98% | Yes |
| 7 | TSS | 18,200 | 3,600 | 80% | Yes |
| | TP | 37 | 11.1 | 70% | Yes |
| | TN | 257 | 122 | 53% | Yes |
| | GP | 3,970 | 170 | 96% | Yes |
| Jetty Road Stage 2 | TSS | 91,300 | 17,000 | 81% | Yes |
| | TP | 186 | 62 | 67% | Yes |

| Outfall | Pollutant | Post-development Inflow Load (kg/year) | Post-development Outflow Load (kg/year) | Load Reduction | BPEM Target Met |
|---|---|---|---|---|---|
| | TN | 1310 | 597 | 54% | Yes |
| | GP | 20,200 | 718 | 96% | Yes |

a. Excluding Outfall 1 catchment loads

It is important to note that water quality objectives are met and exceeded at the combined outfall overall. There may therefore be scope to optimise the size of the water quality assets at a later design stage.

#### 4.2.1 Maintenance

A suitable Maintenance Plan should be developed at a later design stage of the water quality treatment assets. This should take into account the dewatering area and clean out frequency identified in Appendix D.

### 4.3 Volume Assessment

As per the Jetty Road UGP, post-development runoff volume should not exceed the existing conditions runoff volumes to minimise adverse impact on downstream waterways and properties. The average annual runoff volume generated at each outfall under existing and post-development conditions were calculated using MUSIC modelling. For completeness, external catchments draining through Outfalls 1 and 2 were also included in the volume assessment. A summary of runoff volume at each outfall is presented in Table 4-6.

**TABLE 4-6 EXISTING AND POST-DEVELOPMENT VOLUMES AT OUTFALLS (INCLUDING EXTERNAL CATCHMENTS DRAINING THROUGH OUTFALL 1 AND 2)**

| Outfall | Existing Volume (ML/year) | Post Dev with WSUD Volume (ML/year) | Excess volume (ML/year) | Notes |
|---|---|---|---|---|
| 1 | 232 | 318 | | Volume assessment at provided at outfall 2 |
| 2 | 340 | 448 | **108** | Combined outfall 1 and 2 including external catchments.<br>Additional measures needed for volume management. |
| 3 | 10 | 44 | **34** | Piped ocean outfall. Therefore, no impact on downstream waterway or private property. No volume management needed. |
| 4 | 12 | 53 | **42** | Piped ocean outfall. Therefore, no impact on downstream waterway or private property. No volume management needed. |
| 5 | 5 | 26 | **22** | Additional measures needed for volume management. |

| Outfall | Existing Volume (ML/year) | Post Dev with WSUD Volume (ML/year) | Excess volume (ML/year) | Notes |
|---|---|---|---|---|
| 6 | 3 | 35 | **32** | Additional measures needed for volume management. |
| 7 | 11 | 81 | **70** | Piped ocean outfall. Therefore, no impact on downstream waterway or private property. No volume management needed. |

The post-development runoff volume is 181% of the existing conditions, discharging an excess runoff volume of 325 ML/year through seven outfalls. As identified in Section 3.5, maintaining runoff volumes to existing conditions may not be critical at some outfalls. For example, Outfalls 3 and 4 are proposed to be connected to a Council pipe along the Coriyule Road which flows to an ocean outfall. Similarly, due to close proximity of Outfall 7 to Port Phillip Bay, the impact of increased volumes on downstream properties is minimal.

Outfalls 2, 5 and 6 discharge to existing waterways west of the site through private property. Without additional measures such as stormwater harvesting and infiltration within the development, the post-development runoff volume will exceed the pre-development volume. For these outfalls, the proposed strategy aims to maintain post-development runoff volume similar to existing conditions.

A combined total of 161 ML/year excess stormwater runoff volume generated at Outfalls 2, 5, and 6 needs to be retained within the development through stormwater harvesting and infiltration measures. Three options have been investigated to achieve this:

- Option 1 – Stormwater harvesting
  - A sensitivity check was conducted to test the contribution for lot-scale rainwater harvesting. It was assumed each residential lot will be connected to a 2-kL rainwater tank for toilet flushing as per the Jetty Road Urban Growth Plan Flooding, Drainage and Utility Services Principles and Objectives (Objective 27.1). For modelling purposes, it was assumed that only 50% of the residential roof is connected to the rainwater tank (Refer to Appendix E for more details on rainwater harvesting calculations). Modelling results indicate an average of 22 ML/year could be retained through rainwater harvesting.
  - There is still significant runoff volume (139 ML/year) to be managed through other interventions. If the excess runoff volume to be used for open space irrigation, this will be equivalent to irrigating 31 - 44 ha of turfed areas[12]. This option will also require substantial storage to be provided. Curlewis Golf Course may offer an opportunity to provide a potential re-use site. However, it is understood that currently the Curlewis Golf Course use reclaimed water for irrigation of the course. Stormwater could also harvested for reusing at agricultural lands to the west of the current development. These options are to be further investigated at later stages.
- Option 2 – Evaporation Bain
  - If the excess runoff volume of 161 ML/year to be lost via evaporation, MUISC modelling results indicated that an evaporative basin with ~14.5 ha surface area is required.

[12] Based on typical reuse demand of 3.2 and 4.5 ML/ha/year for Warm season turf and Cool season turf (City of Greater Geelong MUSIC modelling guidelines).

- Option 3 – Ocean Outfalls

  Alternatively, there is potential to consider an ocean outfall through the existing council pipe along the Coriyule Road. The feasibility of this option is subject to the ability of the council pipe to receive excess flows. This option is detailed in Section 3.5. Due to significant land take requirements of the other two options considered, Option 3 – ocean outfalls was considered as the preferred volume management strategy for Jetty Road stage 2 development.

# 5 SUMMARY

This report sets out a recommended Stormwater Management Strategy (SWMS) for a proposed residential subdivision of the land known as Jetty Road Stage 2. This report forms part of the DPO application for Jetty Road – Stage 2. The key objective of the SWMS is to identify the key drainage and WSUD assets and their sizing to ensure that the peak discharge rate, volume and pollutant load of stormwater leaving the development area are no greater than pre-development rates. This has done in accordance with the guiding principles of the flooding and drainage management of Jetty Road Urban Growth Plan (UGP)[1] and in line with both City of Greater Geelong and CCMA surface water management requirements.

Seven outfalls along the western boundary of the development have been identified. A small portion of land (24ha) located in the north of the site that is currently draining east is proposed to be diverted to these outfalls. The required 1% AEP flood retardation volumes at outfalls (except Outfall 2 where a no detention strategy is proposed) were calculated along with pipe outlet configurations to maintain post-development peak flows back to existing conditions peak flows. A review of notional risks of the no detention strategy indicated that with the provision of sufficient water quality treatment and erosion control measures, the risks to the downstream waterway and properties can be mitigated.

A WSUD strategy with end-of-pipe treatment asset alone cannot meet the runoff volume management targets. Three potential options were investigated to assist in meeting the requirements:

- A combination of lot-scale rainwater harvesting and precinct scale stormwater harvesting;
- An evaporation basin and
- An ocean outfall.

Due to significant land take requirements of the other two options considered, Ocean outfalls options was considered as the preferred volume management strategy for Jetty Road stage 2 development.

A constructed waterway with a 40 m minimum waterway corridor width (40m or 45m pending on the access provisions from both sides of the waterway) will be required to convey 1% AEP flows through Outfalls 1 and 2, noting the overall waterway channel width is likely to be far less than the 40m corridor width.

Preliminary sizing of WSUD assets required to meet the BPEM water quality targets at each outfall were modelled using MUSIC software and incorporate GPT’s, sedimentation ponds, bioretention and wetlands.

Feasibility of these options to be further investigated at a later design stage. A summary of key drainage assets is presented in Table 5-1.

**TABLE 5-1 DRAINAGE ASSET SUMMARY**

| Outfall | Peak Flow Management (1% AEP Event) | Water Quality Management | Volume Management |
|---|---|---|---|
| 1 | Existing Flow Rate ($m^3/s$) : 1.83<br>Mitigate Flow Rate ($m^3/s$) : 1.71<br>Outlet Pipe : 1 x 750 mm<br>Storage Volume ($m^3$) : 12,800<br>Storage Footprint ($m^2$) : 12,800 | Asset Types : Sed Pond, Wetland<br>Treatment Surface Area ($m^2$) : 13,100<br>Asset Footprint Area ($m^2$) : 26,200 | Volume assessment at provided at outfall 2<br>Combined outfall 1 and 2 including external catchments.<br>Additional measures needed for volume management. Preferred option is an ocean outfall (diverting additional volume to Outfall 3). |
| 2 | Existing Flow Rate ($m^3/s$) : 5.56<br>Mitigate Flow Rate ($m^3/s$) : 5.29<br>Outlet Pipe : -<br>Storage Volume ($m^3$) : -<br>Storage Footprint ($m^2$) : - | Asset Types : Opt 1: Sed Pond, Wetland<br>Opt 2: GPT, Bioretention<br>Treatment Surface Area ($m^2$) : Opt 1: 2,500<br>Opt 2: 900<br>Asset Footprint Area ($m^2$) : Opt 1: 5,500<br>Opt 2: 1,980 | |
| 3 | Existing Flow Rate ($m^3/s$) : 0.78<br>Mitigate Flow Rate ($m^3/s$) : 0.56<br>Outlet Pipe : 1 x 600 mm<br>Storage Volume ($m^3$) : 2,790<br>Storage Footprint ($m^2$) : 4,413 | Asset Types : Sed Pond, Wetland<br>Treatment Surface Area ($m^2$) : 4,150<br>Asset Footprint Area ($m^2$) : 9,130 | Piped outfall.<br>No volume management needed |
| 4 | Existing Flow Rate ($m^3/s$) : 0.95<br>Mitigate Flow Rate ($m^3/s$) : 0.7<br>Outlet Pipe : 1 x 600 mm<br>Storage Volume ($m^3$) : 3,340<br>Storage Footprint ($m^2$) : 4,587 | Asset Types : Sed Pond, Wetland<br>Treatment Surface Area ($m^2$) : 5,100<br>Asset Footprint Area ($m^2$) : 11,220 | Piped outfall.<br>No volume management needed |

| Outfall | Peak Flow Management (1% AEP Event) | Water Quality Management | Volume Management |
|---|---|---|---|
| 5 | Existing Flow Rate ($m^3/s$) : 0.51<br>Mitigate Flow Rate ($m^3/s$) : 0.37<br>Outlet Pipe : 1 x 450 mm<br>Storage Volume ($m^3$) : 1,410<br>Storage Footprint ($m^2$) : 2,440 | Asset Types : Opt 1: Sed Pond, Wetland<br>Opt 2: GPT , Bioretention<br>Treatment Surface Area ($m^2$) : Opt 1: 2,850<br>Opt 2: 700<br>Asset Footprint Area ($m^2$) : Opt 1: 6,270<br>Opt 2: 1,540 | Additional measures needed for volume management. Preferred option is an ocean outfall (diverting additional volume to Outfall 4). |
| 6 | Existing Flow Rate ($m^3/s$) : 0.4<br>Mitigate Flow Rate ($m^3/s$) : 0.19<br>Outlet Pipe : 1 x 300 mm<br>Storage Volume ($m^3$) : 3,200<br>Storage Footprint ($m^2$) : 3,969 | Asset Types : Sed Pond, Wetland<br>Treatment Surface Area ($m^2$) : 3,400<br>Asset Footprint Area ($m^2$) : 7,480 | Additional measures needed for volume management. Preferred option is an ocean outfall (diverting additional volume to Outfall 7). |
| 7 | Existing Flow Rate ($m^3/s$) : 0.3<br>Mitigate Flow Rate ($m^3/s$) : 0.25<br>Outlet Pipe : 1 x 300 mm<br>Storage Volume ($m^3$) : 10,300<br>Storage Footprint ($m^2$) : 7,950 | Asset Types : Sed Pond, Wetland<br>Treatment Surface Area ($m^2$) : 7,900<br>Asset Footprint Area ($m^2$) : 17,380 | Piped outfall.<br>No volume management needed |

# APPENDIX A
# RORB MODEL SETUP

# A-1 Existing Conditions

## A-1-1 Subareas

**TABLE A-1 EXISTING CONDTIONS SUBAREAS**

| Subarea Name | Area (km$^2$) | TIA | PA | EIA | ICA |
|---|---|---|---|---|---|
| B | 0.073324 | 0.05 | 0.95 | 0.00 | 0.05 |
| C | 0.098997 | 0.05 | 0.95 | 0.00 | 0.05 |
| D | 0.042534 | 0.05 | 0.95 | 0.00 | 0.05 |
| E | 0.1121 | 0.05 | 0.95 | 0.00 | 0.05 |
| F | 0.088025 | 0.05 | 0.95 | 0.00 | 0.05 |
| G | 0.053866 | 0.75 | 0.25 | 0.45 | 0.30 |
| H | 0.02302 | 0.05 | 0.95 | 0.00 | 0.05 |
| I | 0.047751 | 0.75 | 0.25 | 0.45 | 0.30 |
| J | 0.055774 | 0.75 | 0.25 | 0.45 | 0.30 |
| K | 0.08047 | 0.75 | 0.25 | 0.45 | 0.30 |
| L | 0.033903 | 0.75 | 0.25 | 0.45 | 0.30 |
| M | 0.046561 | 0.75 | 0.25 | 0.45 | 0.30 |
| N | 0.019024 | 0.75 | 0.25 | 0.45 | 0.30 |
| O | 0.082042 | 0.29 | 0.71 | 0.17 | 0.12 |
| P | 0.168116 | 0.19 | 0.81 | 0.11 | 0.08 |
| Q | 0.267196 | 0.07 | 0.93 | 0.04 | 0.03 |
| R | 0.113748 | 0.20 | 0.80 | 0.12 | 0.08 |
| S | 0.135264 | 0.06 | 0.94 | 0.04 | 0.03 |
| T | 0.1842 | 0.05 | 0.95 | 0.00 | 0.05 |
| U | 0.153815 | 0.05 | 0.95 | 0.00 | 0.05 |
| V | 0.250462 | 0.05 | 0.95 | 0.00 | 0.05 |
| W | 0.344965 | 0.05 | 0.95 | 0.00 | 0.05 |
| X | 0.342861 | 0.05 | 0.95 | 0.00 | 0.05 |
| Y | 0.143631 | 0.05 | 0.95 | 0.00 | 0.05 |
| Z | 0.430177 | 0.05 | 0.95 | 0.00 | 0.05 |
| AA | 0.015786 | 0.05 | 0.95 | 0.00 | 0.05 |
| AB | 0.010552 | 0.05 | 0.95 | 0.00 | 0.05 |
| AC | 0.014672 | 0.05 | 0.95 | 0.00 | 0.05 |
| AD | 0.010068 | 0.05 | 0.95 | 0.00 | 0.05 |
| AE | 0.038725 | 0.05 | 0.95 | 0.00 | 0.05 |
| AF | 0.025256 | 0.05 | 0.95 | 0.00 | 0.05 |
| AG | 0.047831 | 0.05 | 0.95 | 0.00 | 0.05 |
| AH | 0.047041 | 0.05 | 0.95 | 0.00 | 0.05 |
| AI | 0.019401 | 0.05 | 0.95 | 0.00 | 0.05 |
| AJ | 0.025889 | 0.05 | 0.95 | 0.00 | 0.05 |
| AK | 0.023558 | 0.05 | 0.95 | 0.00 | 0.05 |

| Subarea Name | Area (km$^2$) | TIA | PA | EIA | ICA |
|---|---|---|---|---|---|
| AL | 0.038614 | 0.05 | 0.95 | 0.00 | 0.05 |
| AM | 0.034331 | 0.05 | 0.95 | 0.00 | 0.05 |
| AN | 0.02392 | 0.05 | 0.95 | 0.00 | 0.05 |
| AO | 0.013472 | 0.05 | 0.95 | 0.00 | 0.05 |
| AP | 0.029913 | 0.05 | 0.95 | 0.00 | 0.05 |
| AQ | 0.018723 | 0.05 | 0.95 | 0.00 | 0.05 |
| AR | 0.009936 | 0.05 | 0.95 | 0.00 | 0.05 |
| AS | 0.011327 | 0.05 | 0.95 | 0.00 | 0.05 |
| AT | 0.016494 | 0.05 | 0.95 | 0.00 | 0.05 |
| AU | 0.012941 | 0.05 | 0.95 | 0.00 | 0.05 |
| AV | 0.031606 | 0.05 | 0.95 | 0.00 | 0.05 |
| AW | 0.366198 | 0.05 | 0.95 | 0.00 | 0.05 |
| AX | 0.085852 | 0.05 | 0.95 | 0.00 | 0.05 |
| AY | 0.037513 | 0.70 | 0.30 | 0.42 | 0.28 |
| AZ | 0.053518 | 0.75 | 0.25 | 0.45 | 0.30 |
| BA | 0.079295 | 0.75 | 0.25 | 0.45 | 0.30 |
| BB | 0.00897 | 0.05 | 0.95 | 0.00 | 0.05 |
| BC | 0.018372 | 0.05 | 0.95 | 0.00 | 0.05 |
| BD | 0.713919 | 0.06 | 0.94 | 0.00 | 0.06 |
| BF | 0.011927 | 0.20 | 0.80 | 0.12 | 0.08 |
| BG | 0.032233 | 0.20 | 0.80 | 0.12 | 0.08 |
| BH | 0.041643 | 0.20 | 0.80 | 0.12 | 0.08 |
| BI | 0.058634 | 0.20 | 0.80 | 0.12 | 0.08 |
| BJ | 0.077625 | 0.20 | 0.80 | 0.12 | 0.08 |
| BK | 0.059255 | 0.20 | 0.80 | 0.12 | 0.08 |
| BL | 0.07001 | 0.20 | 0.80 | 0.12 | 0.08 |
| BM | 0.013838 | 0.10 | 0.90 | 0.06 | 0.04 |
| BN | 0.063949 | 0.20 | 0.80 | 0.12 | 0.08 |
| BO | 0.009031 | 0.05 | 0.95 | 0.00 | 0.05 |
| BP | 0.009974 | 0.05 | 0.95 | 0.00 | 0.05 |
| BQ | 0.011429 | 0.05 | 0.95 | 0.00 | 0.05 |
| BR | 0.01101 | 0.05 | 0.95 | 0.00 | 0.05 |
| BS | 0.010613 | 0.05 | 0.95 | 0.00 | 0.05 |
| BT | 0.004271 | 0.05 | 0.95 | 0.00 | 0.05 |
| BU | 0.033319 | 0.05 | 0.95 | 0.00 | 0.05 |
| BV | 0.020795 | 0.05 | 0.95 | 0.00 | 0.05 |
| BW | 0.01197 | 0.05 | 0.95 | 0.00 | 0.05 |
| BX | 0.057568 | 0.05 | 0.95 | 0.00 | 0.05 |
| BY | 0.028743 | 0.75 | 0.25 | 0.45 | 0.30 |
| BZ | 0.027997 | 0.75 | 0.25 | 0.45 | 0.30 |
| CA | 0.083416 | 0.75 | 0.25 | 0.45 | 0.30 |
| CB | 0.039687 | 0.05 | 0.95 | 0.03 | 0.02 |

| Subarea Name | Area (km$^2$) | TIA | PA | EIA | ICA |
|---|---|---|---|---|---|
| CC | 0.02043 | 0.05 | 0.95 | 0.00 | 0.05 |
| CD | 0.018724 | 0.05 | 0.95 | 0.00 | 0.05 |
| CE | 0.015551 | 0.05 | 0.95 | 0.00 | 0.05 |
| CF | 0.01853 | 0.05 | 0.95 | 0.00 | 0.05 |
| CG | 0.067674 | 0.05 | 0.95 | 0.00 | 0.05 |
| CH | 0.050713 | 0.05 | 0.95 | 0.00 | 0.05 |
| CI | 0.028248 | 0.05 | 0.95 | 0.00 | 0.05 |
| CJ | 0.029471 | 0.05 | 0.95 | 0.00 | 0.05 |
| CK | 0.111421 | 0.05 | 0.95 | 0.00 | 0.05 |
| CL | 0.069338 | 0.05 | 0.95 | 0.00 | 0.05 |
| CM | 0.124868 | 0.05 | 0.95 | 0.00 | 0.05 |
| CN | 0.025244 | 0.05 | 0.95 | 0.00 | 0.05 |
| CO | 0.011039 | 0.10 | 0.90 | 0.06 | 0.04 |
| CP | 0.162682 | 0.75 | 0.25 | 0.45 | 0.30 |
| CQ | 0.110533 | 0.70 | 0.30 | 0.42 | 0.28 |
| CR | 0.200167 | 0.70 | 0.30 | 0.42 | 0.28 |
| CS | 0.576951 | 0.72 | 0.28 | 0.43 | 0.29 |
| CU | 0.374524 | 0.05 | 0.95 | 0.03 | 0.02 |
| CV | 0.495865 | 0.07 | 0.93 | 0.04 | 0.03 |
| CW | 0.528323 | 0.24 | 0.76 | 0.14 | 0.09 |
| CX | 0.210404 | 0.11 | 0.89 | 0.07 | 0.04 |
| CY | 0.705647 | 0.28 | 0.72 | 0.17 | 0.11 |
| CZ | 0.388886 | 0.72 | 0.28 | 0.43 | 0.29 |
| DA | 0.922702 | 0.22 | 0.78 | 0.13 | 0.09 |
| DB | 0.610711 | 0.54 | 0.46 | 0.33 | 0.22 |
| DC | 0.194088 | 0.37 | 0.63 | 0.22 | 0.15 |
| DD | 1.191282 | 0.60 | 0.40 | 0.36 | 0.24 |
| DE | 0.404029 | 0.75 | 0.25 | 0.45 | 0.30 |
| DF | 0.654401 | 0.71 | 0.29 | 0.43 | 0.28 |
| A | 0.424936 | 0.53 | 0.47 | 0.32 | 0.21 |
| DG | 0.46208 | 0.05 | 0.95 | 0.00 | 0.05 |
| CT | 0.311235 | 0.06 | 0.94 | 0.04 | 0.02 |
| CT2 | 0.189973 | 0.55 | 0.45 | 0.33 | 0.22 |
| BE | 0.183353 | 0.07 | 0.93 | 0.04 | 0.03 |
| BE2 | 0.129359 | 0.18 | 0.82 | 0.11 | 0.07 |

Subareas marked in red are rural and do not have directly connected impervious area

## A-1-2 Existing basins

TABLE A-2 STRAMPTON AVENUE BASIN (BELLAVIEW ESTATE)

| Stage | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 0 | 0 | 0 |
| 0.21 | 59.96 | 0.0176 |
| 0.49 | 176.30 | 0.1339 |
| 0.5 | 181.80 | 0.1381 |
| 0.88 | 463.57 | 0.2959 |
| 1.46 | 1083.30 | 0.5368 |
| 1.64 | 1321.71 | 1.0896 |
| 2.64 | 3043.71 | 2.3791 |

TABLE A-3 GREENVALE BASIN (BELLAVIEW ESTATE)

| Stage | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 0 | 0 | 0 |
| 0.11 | 74.80 | 0.06 |
| 0.31 | 240.80 | 0.19 |
| 0.95 | 976.80 | 0.59 |
| 1.46 | 1801.47 | 0.92 |
| 1.56 | 1987.92 | 1.62 |
| 1.86 | 2573.67 | 4.94 |
| 2.86 | 4732.17 | 16.00 |

TABLE A-4 APPLEBY BASIN (CURLEWIS PARK ESTATE)

| Stage | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 0 | 0 | 0 |
| 0.65 | 548.60 | 0.22 |
| 1 | 975.25 | 0.33 |
| 1.3 | 1417.75 | 1.44 |
| 4.2 | 9562.40 | 6.64 |

**TABLE A-5 SMEC NORTH BASIN (CURLEWIS PARK ESTATE)**

| Stage | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 0 | 0 | 0 |
| 0.25 | 270.88 | 0.20 |
| 0.85 | 1138.31 | 0.43 |
| 1.15 | 1834.91 | 0.49 |
| 1.35 | 2450.28 | 0.53 |
| 1.7 | 3656.27 | 0.59 |
| 2 | 4808.33 | 0.65 |

**TABLE A-6 SMEC SOUTH BASIN (CURLEWIS PARK ESTATE)**

| Stage | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 0 | 0 | 0 |
| 0.52 | 143.43 | 0.012 |
| 1.03 | 422.04 | 0.02 |
| 1.7 | 1209.63 | 0.02 |
| 2.5 | 2673.23 | 34.84 |
| 2.85 | 3399.30 | 50.14 |

Low flows are diverted from SMEC South basin to SMEC north basin via a 150 mm pipe. This diversion was represented using Inflow/Outflow nodes in RORB model. Diversion was represented as a function of SMEC south basin outflow.

**TABLE A-7 DIVERSION OF LOW-FLOWS INTO SMEC NORTH BASIN**

| Basin Outflow ($m^3/s$) | Diversion ($m^3/s$) |
|---|---|
| 0 | 0 |
| 0.02 | 0.02 |
| 34.84 | 0.02 |
| 50.15 | 0.02 |

**TABLE A-8 B1 (DRYSDALE BYPASS)**

| Height (m) | Storage ($m^3$) |
|---|---|
| 0 | 0 |
| 5 | 59,125 |

Outlet consist of 450 mm pipe at 1% slope

TABLE A-9 B2 (DRYSDALE BYPASS)

| Height (m) | Storage ($m^3$) |
|---|---|
| 0 | 0 |
| 6 | 18,000 |

Outlet consist of single 525 mm diameter pipe at 1% slope

TABLE A-10 GOLF CLUB CULVERT CROSSING MODELLED AS A STORAGE

| Stage (mAHD) | Storage ($m^3$) | Discharge ($m^3/s$) |
|---|---|---|
| 45.11 | 0 | 0.00 |
| 45.69 | 22 | 0.80 |
| 46.04 | 45 | 1.60 |
| 46.36 | 64 | 2.40 |
| 46.85 | 98 | 3.20 |
| 47.39 | 188 | 4.00 |
| 48.08 | 434 | 4.80 |
| 49.00 | 1,130 | 5.60 |
| 50.05 | 2,193 | 6.40 |
| 51.06 | 3,779 | 7.55 |
| 51.21 | 4,030 | 10.26 |
| 52.00 | 8,056 | 39.19 |

- Stage storage was derived using LiDAR data.
- Culvert crossing details were extracted from survey data provided by SMEC (1421 Portalington Road - Ver A.dwg, received on 29 March 2022)
- Stage-discharge relationship was determined using HY-8 model for the RCBC.

## A-2 Post-development Conditions

### A-2-1 Subareas

TABLE A-11 WITH DIFFERENT IMPERVIOUS FRACTIONS IN THE DEVELOPED MODEL

| Subarea Name | Area ($km^2$) | TIA-Ext | TIA-PostDev | PA-PostDev | EIA-PostDev | ICA-PostDev |
|---|---|---|---|---|---|---|
| H | 0.023 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| R | 0.114 | 0.20 | 0.70 | 0.30 | 0.42 | 0.28 |
| AA | 0.016 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AB | 0.011 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AC | 0.015 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AD | 0.01 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AE | 0.039 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AF | 0.025 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AG | 0.048 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AH | 0.047 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AI | 0.019 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AJ | 0.026 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AK | 0.024 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AL | 0.039 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AM | 0.034 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AN | 0.024 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AO | 0.013 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AP | 0.03 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AQ | 0.019 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AR | 0.01 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AS | 0.011 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AT | 0.016 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| AU | 0.013 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BB | 0.009 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BC | 0.018 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BE | 0.309 | 0.07 | 0.12 | 0.88 | 0.07 | 0.05 |
| BF | 0.012 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BG | 0.032 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BH | 0.042 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BI | 0.059 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BJ | 0.078 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BK | 0.059 | 0.20 | 0.75 | 0.25 | 0.45 | 0.30 |
| BL | 0.07 | 0.20 | 0.74 | 0.26 | 0.44 | 0.30 |
| BN | 0.064 | 0.20 | 0.73 | 0.27 | 0.44 | 0.29 |
| BO | 0.009 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BP | 0.01 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |

| Subarea Name | Area ($km^2$) | TIA-Ext | TIA-PostDev | PA-PostDev | EIA-PostDev | ICA-PostDev |
|---|---|---|---|---|---|---|
| BQ | 0.011 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BR | 0.011 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BS | 0.011 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BT | 0.004 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BU | 0.033 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BV | 0.021 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BW | 0.012 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| BX | 0.058 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CC | 0.02 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CD | 0.019 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CE | 0.016 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CF | 0.019 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CG | 0.068 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CH | 0.051 | 0.05 | 0.75 | 0.25 | 0.45 | 0.30 |
| CT | 0.505 | 0.06 | 0.57 | 0.43 | 0.34 | 0.23 |

## A-2-2 Basin Summary

All basins were modelled assuming trapezoidal storage with 1 in 6 batters
All outfall pipes were set at 1% slope
Pipe entrance loss coefficient taken as 0.5

**TABLE A-12 PROPOSED BASIN SUMMARY**

| Outfall/RB | Base Length (m) | Base Width (m) | 1% AEP Peak Elevation | Outlet pipe size |
|---|---|---|---|---|
| RB1 | 100 | 45 | 1.99 | 1 x 750 mm |
| RB3 | 85 | 30 | 0.87 | 1 x 600 mm |
| RB3 – Ocean outfall | 85 | 30 | 0.89 | 1 x 600 mm |
| RB4 | 85 | 30 | 1.01 | 1 x 600 mm |
| RB4 – Ocean outfall | 102 | 45 | 0.89 | 1 x 600 mm |
| RB5 | 35 | 35 | 0.86 | 1 x 450 mm |
| RB6 | 45 | 45 | 1.17 | 1 x 300 mm |
| RB7 | 90 | 40 | 1.95 | 1 x 300 mm |
| RB7 – Ocean outfall | 135 | 60 | 1.69 | 1 x 300 mm |

# A-3 RORB Model Sensitivity Check

Design parameters of the two Drysdale Bypass basins (Jetty Road Basin and Grubbs Road Basin) were not available at the time of original RORB modelling was undertaken. Upon Council review of the draft SWMP, as-constructed drawings and the detailed design report was made available. A sensitivity check was undertaken by updating the basin details in the existing scenario RORB model. This section provides a summary of model changes.

**TABLE A-13 BASIN B1 (GRUBBS ROAD BASIN)**

| Model Parameter | Original SWMP Model | Sensitivity Check |
|---|---|---|
| Max Height (m) | 5 | 1.3 |
| Max Volume ($m^3$) | 59,125 | 15,987 |
| Outlet pipe<br>• Length (m)<br>• Slope (%)<br>• No. of pipes<br>• Pipe diameter (m) | 50<br>1<br>1<br>0.450 | 127<br>0.4<br>1<br>0.450 |
| Spillway<br>• Crest (m)<br>• Width (m) | n/a | 1.3<br>5 |

**TABLE A-14 BASIN B2 (JETTY ROAD BASIN)**

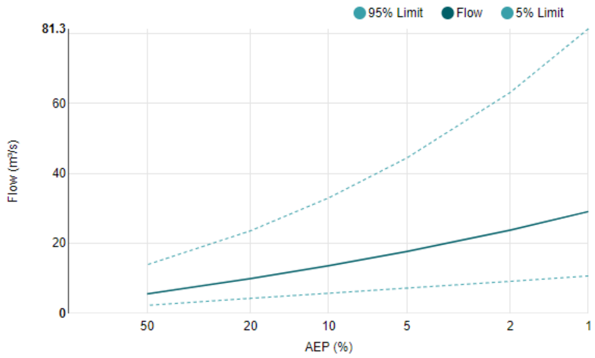
| Model Parameter | Original SWMP Model | Sensitivity Check |
|---|---|---|
| Max Height (m) | 6 | 3.4 |
| Max Volume ($m^3$) | 18,000 | 8,000 |
| Outlet pipe<br>• Length (m)<br>• Slope (%)<br>• No. of pipes<br>• Pipe diameter (m) | 25<br>1<br>1<br>0.525 | 16<br>1.71<br>1<br>.525 |
| Spillway<br>• Crest (m)<br>• Width (m) | n/a | 3.4<br>5 |

TABLE A-15 MODEL RESUTLS COMPARISON - 1% AEP EVENT EXISTING CONDITIONS

| Model Parameter | Original SWMP Model | Sensitivity Check |
|---|---|---|
| Basin B1<br>• Peak flood storage ($m^3$)<br>• Peak outflow rate ($m^3/s$) | <br>21,100<br>0.76 | <br>23,700<br>0.43 |
| Basin B2<br>• Peak flood storage ($m^3$)<br>• Peak outflow rate ($m^3/s$) | <br>5,500<br>0.81 | <br>3,960<br>0.82 |
| Peak flow rate at the Jetty Road ($m^3/s$) | 0.81 | 0.82 |
| Peak flow rate at the Outfall 1 ($m^3/s$) | 1.83 | 1.80 |
| Peak flow rate at the Outfall 2 ($m^3/s$) | 5.56 | 5.52 |

# APPENDIX B
# REGIONAL FLOOD FREQUENCY ESTIMATE

# Results | Regional Flood Frequency Estimation Model




| AEP (%) | Discharge ($m^3/s$) | Lower Confidence Limit (5%) ($m^3/s$) | Upper Confidence Limit (95%) ($m^3/s$) |
|---|---|---|---|
| 50 | 5.58 | 2.30 | 13.9 |
| 20 | 9.96 | 4.31 | 23.6 |
| 10 | 13.6 | 5.76 | 33.0 |
| 5 | 17.7 | 7.24 | 44.4 |
| 2 | 23.8 | 9.16 | 63.1 |
| 1 | 29.1 | 10.7 | 81.3 |

| Input Data | |
|---|---|
| Date/Time | 2022-05-19 07:00 |
| Catchment Name | Catchment1 |
| Latitude (Outlet) | -38.16032 |
| Longitude (Outlet) | 144.51944 |
| Latitude (Centroid) | -38.17485 |
| Longitude (Centroid) | 144.5518 |
| Catchment Area ($km^2$) | 16.085 |
| Distance to Nearest Gauged Catchment (km) | 28.34 |
| 50% AEP 6 Hour Rainfall Intensity (mm/h) | 4.513329 |
| 2% AEP 6 Hour Rainfall Intensity (mm/h) | 9.873671 |
| Rainfall Intensity Source (User/Auto) | Auto |
| Region | East Coast |
| Region Version | RFFE Model 2016 v1 |
| Region Source (User/Auto) | Auto |

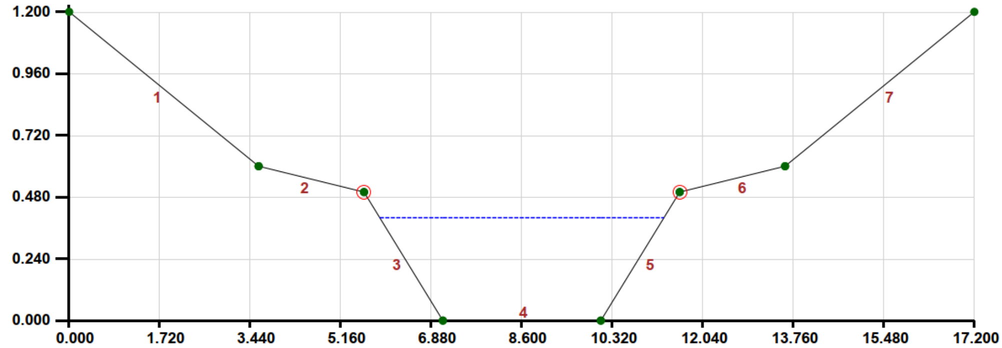
# APPENDIX C
# PC CONVEY ANALYSIS - WATERWAY

# C-1 Outfall 1

## C-1-1 LFC Analysis

PROJECT: Jetty Road Stage 2
CWD XS (Outfall 1)
Print-out date: 27/05/2022 - Time: 1:39
Data File: Constructed_Waterway_Outfall1.dat

1. CROSS-SECTION




2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 1,050 litres/second

3. RESULTS Water surface elevation = 0.400 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

| | LEFT OVERBANK | MAIN CHANNEL | RIGHT OVERBANK | TOTAL CROSS-SECTION |
|---|---|---|---|---|
| Discharge (litres/second): | 0.000 | 1,073.717 | 0.000 | 1,073.717 |
| D(Max) = Max. Depth (m): | 0.000 | 0.400 | 0.000 | 0.400 |
| D(Ave) = Ave. Depth (m): | 0.000 | 0.311 | 0.000 | 0.311 |
| V = Ave. Velocity (m/s): | 0.000 | 0.639 | 0.000 | 0.639 |
| D(Max) x V (cumecs/m): | 0.000 | 0.256 | 0.000 | 0.256 |
| D(Ave) x V (cumecs/m): | 0.000 | 0.199 | 0.000 | 0.199 |
| Froude Number: | 0.000 | 0.366 | 0.000 | 0.366 |
| Area (m^2): | 0.000 | 1.680 | 0.000 | 1.680 |
| Wetted Perimeter (m): | 0.000 | 5.530 | 0.000 | 5.530 |
| Flow Width (m): | 0.000 | 5.400 | 0.000 | 5.400 |
| Hydraulic Radius (m): | 0.000 | 0.304 | 0.000 | 0.304 |
| Composite Manning's n: | 0.000 | 0.050 | 0.000 | 0.050 |
| Split Flow? | - | - | - | No |

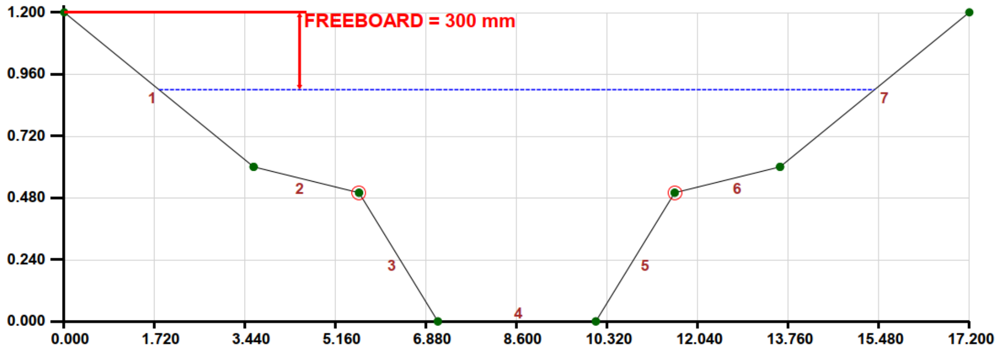
4. CROSS-SECTION DATA

| | LEFT HAND POINT | | RIGHT HAND POINT | | |
|---|---|---|---|---|---|
| SEGMENT NO. | CHAINAGE (m) | R.L. (m) | CHAINAGE (m) | R.L. (m) | MANNING'S N |
| 1 | 0.000 | 1.200 | 3.600 | 0.600 | 0.050 |
| 2 | 3.600 | 0.600 | 5.600 | 0.500 | 0.050 |
| 3 | 5.600 | 0.500 | 7.100 | 0.000 | 0.050 |
| 4 | 7.100 | 0.000 | 10.100 | 0.000 | 0.050 |
| 5 | 10.100 | 0.000 | 11.600 | 0.500 | 0.050 |
| 6 | 11.600 | 0.500 | 13.600 | 0.600 | 0.050 |
| 7 | 13.600 | 0.600 | 17.200 | 1.200 | 0.050 |

## C-1-2 HFC Analysis

PROJECT: Jetty Road Stage 2
CWD XS
Print-out date: 27/05/2022 - Time: 1:19
Data File: Constructed_Waterway_Outfall2.dat

1. CROSS-SECTION




2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 6.4 cumecs

3. RESULTS Water surface elevation = 0.900 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

| | LEFT OVERBANK | MAIN CHANNEL | RIGHT OVERBANK | TOTAL CROSS-SECTION |
|---|---|---|---|---|
| Discharge (cumecs): | 0.549 | 5.451 | 0.549 | 6.549 |
| D(Max) = Max. Depth (m): | 0.400 | 0.900 | 0.400 | 0.900 |
| D(Ave) = Ave. Depth (m): | 0.255 | 0.775 | 0.255 | 0.775 |
| V = Ave. Velocity (m/s): | 0.566 | 1.172 | 0.566 | 0.994 |
| D(Max) x V (cumecs/m): | 0.227 | 1.055 | 0.227 | 0.894 |
| D(Ave) x V (cumecs/m): | 0.145 | 0.908 | 0.145 | 0.770 |
| Froude Number: | 0.358 | 0.425 | 0.358 | 0.396 |
| Area (m^2): | 0.970 | 4.650 | 0.970 | 6.590 |
| Wetted Perimeter (m): | 3.827 | 6.162 | 3.827 | 13.817 |
| Flow Width (m): | 3.800 | 6.000 | 3.800 | 13.600 |
| Hydraulic Radius (m): | 0.253 | 0.755 | 0.253 | 0.477 |
| Composite Manning's n: | 0.050 | 0.050 | 0.050 | 0.050 |
| Split Flow? | - | - | - | No |

4. CROSS-SECTION DATA

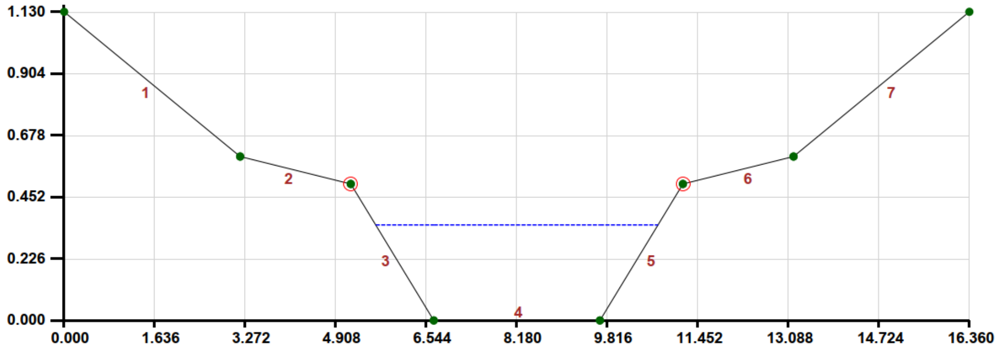
| | LEFT HAND POINT | | RIGHT HAND POINT | | |
|---|---|---|---|---|---|
| SEGMENT NO. | CHAINAGE (m) | R.L. (m) | CHAINAGE (m) | R.L. (m) | MANNING'S N |
| 1 | 0.000 | 1.200 | 3.600 | 0.600 | 0.050 |
| 2 | 3.600 | 0.600 | 5.600 | 0.500 | 0.050 |
| 3 | 5.600 | 0.500 | 7.100 | 0.000 | 0.050 |
| 4 | 7.100 | 0.000 | 10.100 | 0.000 | 0.050 |
| 5 | 10.100 | 0.000 | 11.600 | 0.500 | 0.050 |
| 6 | 11.600 | 0.500 | 13.600 | 0.600 | 0.050 |
| 7 | 13.600 | 0.600 | 17.200 | 1.200 | 0.050 |

# C-2 Outfall 2

## C-2-1 LFC Analysis

PROJECT: Jetty Road Stage 2
CWD XS (Outfall 2)
Print-out date: 27/05/2022 - Time: 7:19
Data File: Constructed_Waterway_Outfall2.dat

1. CROSS-SECTION




2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 800 litres/second

3. RESULTS Water surface elevation = 0.350 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

| | LEFT OVERBANK | MAIN CHANNEL | RIGHT OVERBANK | TOTAL CROSS-SECTION |
|---|---|---|---|---|
| Discharge (litres/second): | 0.000 | 841.321 | 0.000 | 841.321 |
| D(Max) = Max. Depth (m): | 0.000 | 0.350 | 0.000 | 0.350 |
| D(Ave) = Ave. Depth (m): | 0.000 | 0.278 | 0.000 | 0.278 |
| V = Ave. Velocity (m/s): | 0.000 | 0.594 | 0.000 | 0.594 |
| D(Max) x V (cumecs/m): | 0.000 | 0.208 | 0.000 | 0.208 |
| D(Ave) x V (cumecs/m): | 0.000 | 0.165 | 0.000 | 0.165 |
| Froude Number: | 0.000 | 0.359 | 0.000 | 0.359 |
| Area (m^2): | 0.000 | 1.418 | 0.000 | 1.418 |
| Wetted Perimeter (m): | 0.000 | 5.214 | 0.000 | 5.214 |
| Flow Width (m): | 0.000 | 5.100 | 0.000 | 5.100 |
| Hydraulic Radius (m): | 0.000 | 0.272 | 0.000 | 0.272 |
| Composite Manning's n: | 0.000 | 0.050 | 0.000 | 0.050 |
| Split Flow? | - | - | - | No |

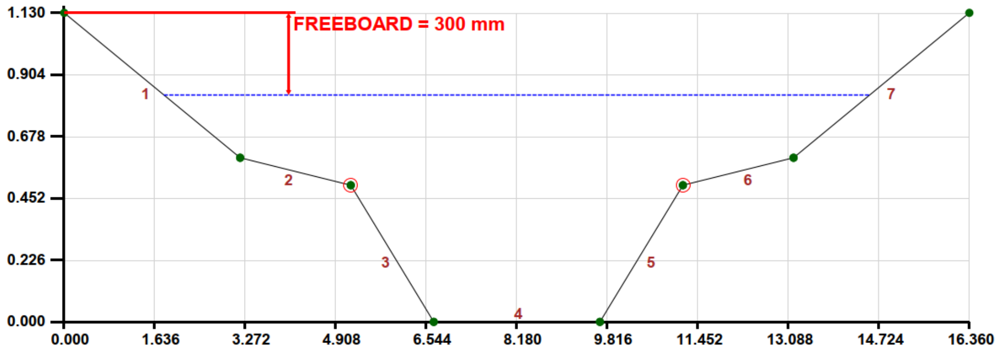
4. CROSS-SECTION DATA

| | LEFT HAND POINT | | RIGHT HAND POINT | | |
|---|---|---|---|---|---|
| SEGMENT NO. | CHAINAGE (m) | R.L. (m) | CHAINAGE (m) | R.L. (m) | MANNING'S N |
| 1 | 0.000 | 1.130 | 3.180 | 0.600 | 0.050 |
| 2 | 3.180 | 0.600 | 5.180 | 0.500 | 0.050 |
| 3 | 5.180 | 0.500 | 6.680 | 0.000 | 0.050 |
| 4 | 6.680 | 0.000 | 9.680 | 0.000 | 0.050 |
| 5 | 9.680 | 0.000 | 11.180 | 0.500 | 0.050 |
| 6 | 11.180 | 0.500 | 13.180 | 0.600 | 0.050 |
| 7 | 13.180 | 0.600 | 16.360 | 1.130 | 0.050 |

# C-2-2 HFC Analysis

PROJECT: Jetty Road Stage 2
CWD XS (Outfall 2)
Print-out date: 27/05/2022 - Time: 7:17
Data File: Constructed_Waterway_Outfall2.dat

1. CROSS-SECTION




2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 5,300 litres/second

3. RESULTS Water surface elevation = 0.830 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

| | LEFT OVERBANK | MAIN CHANNEL | RIGHT OVERBANK | TOTAL CROSS-SECTION |
|---|---|---|---|---|
| Discharge (litres/second): | 360.560 | 4,655.019 | 360.560 | 5,376.139 |
| D(Max) = Max. Depth (m): | 0.330 | 0.830 | 0.330 | 0.830 |
| D(Ave) = Ave. Depth (m): | 0.213 | 0.705 | 0.213 | 0.705 |
| V = Ave. Velocity (m/s): | 0.502 | 1.100 | 0.502 | 0.949 |
| D(Max) x V (cumecs/m): | 0.166 | 0.913 | 0.166 | 0.787 |
| D(Ave) x V (cumecs/m): | 0.107 | 0.776 | 0.107 | 0.669 |
| Froude Number: | 0.347 | 0.418 | 0.347 | 0.390 |
| Area (m^2): | 0.719 | 4.230 | 0.719 | 5.667 |
| Wetted Perimeter (m): | 3.402 | 6.162 | 3.402 | 12.965 |
| Flow Width (m): | 3.380 | 6.000 | 3.380 | 12.760 |
| Hydraulic Radius (m): | 0.211 | 0.686 | 0.211 | 0.437 |
| Composite Manning's n: | 0.050 | 0.050 | 0.050 | 0.050 |
| Split Flow? | - | - | - | No |

4. CROSS-SECTION DATA

| | LEFT HAND POINT | | RIGHT HAND POINT | | |
|---|---|---|---|---|---|
| SEGMENT NO. | CHAINAGE (m) | R.L. (m) | CHAINAGE (m) | R.L. (m) | MANNING'S N |
| 1 | 0.000 | 1.130 | 3.180 | 0.600 | 0.050 |
| 2 | 3.180 | 0.600 | 5.180 | 0.500 | 0.050 |
| 3 | 5.180 | 0.500 | 6.680 | 0.000 | 0.050 |
| 4 | 6.680 | 0.000 | 9.680 | 0.000 | 0.050 |
| 5 | 9.680 | 0.000 | 11.180 | 0.500 | 0.050 |
| 6 | 11.180 | 0.500 | 13.180 | 0.600 | 0.050 |
| 7 | 13.180 | 0.600 | 16.360 | 1.130 | 0.050 |

# APPENDIX D
# SEDIMENT POND SIZING

## Fair and Geyer Equation – Equ 10.3 WSUD Stormwater Technical Manual (2005)

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$$

$$\lambda = 1 - 1/n; \qquad n = \frac{1}{1-\lambda}$$

R = fraction of Initial Solids Removed = 80 - 90 % typ.

| | | |
|---|---|---|
| R | = fraction of Initial Solids Removed = 80 - 90 % typ. | |
| $d_p$ | = Depth of permanent pool | |
| $d_e$ | = Extended detention depth above permanent pool | |
| $d^*$ | = depth below permanent pool sufficient to retain particles (lower of 1.0m or $d_p$) | |
| Q | = design flow (Typically 3 month, 6 month or 1 year flow) | |
| A | = Basin Surface Area | |
| n | = turbulence parameter (see above) | = 1 for significant short circuiting and turbulence<br>= 5 for insignificant short circuiting and turbulence |
| $v_s$ | = setting velocity for particles | |

Table 7.2 Settling velocities under ideal conditions (Maryland Department of Environment, 1987)

| Classification of Particle size range | Particle diameter (µm) | Settling velocities (mm/s) |
|---|---|---|
| Very coarse sand | 2000 | 200 |
| Coarse sand | 1000 | 100 |
| Medium sand | 500 | 53 |
| Fine sand | 250 | 26 |
| Very fine sand | 125 | 11 |
| Coarse silt | 62 | 2.3 |
| Medium silt | 31 | 0.66 |
| Fine silt | 16 | 0.18 |
| Very fine silt | 8 | 0.04 |
| Clay | 4 | 0.011 |

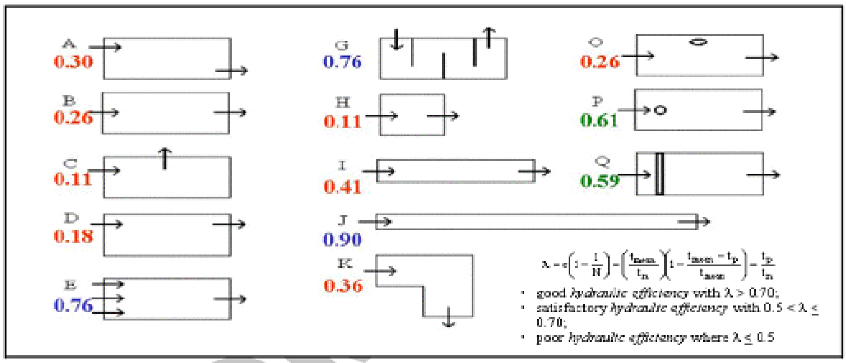



Figure 10.5 Hydraulic Efficiency – A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment [Source: Australian Runoff Quality, 2003]

*Source: WSUD Engineering Procedures: Stormwater Technical Manual DRAFT 2004*

# D-1 Outfall 1

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential developments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| $d^*$ = | 1 | m | (lower of 1 m and dp) |
| $\frac{(d_e+d_p)}{(d_e+d^*)}$ = | 1.38 | | |
| Q = | 1.05 | $m^3/s$ | 1EY flows from RORB modelling |
| A = | **1100** | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $\frac{V_s}{Q/A}$ = | 11.52 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

R = **95.30%**

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

## Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 52.9 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3$/ha/yr | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3$/ha/yr | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 578.42 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3)}{\text{ActualBasinVolume } (m^3)}$ 0.18 per year

Clean out every 5.47 years

***Try to minimise cleanouts - ideally, once every 5 years***

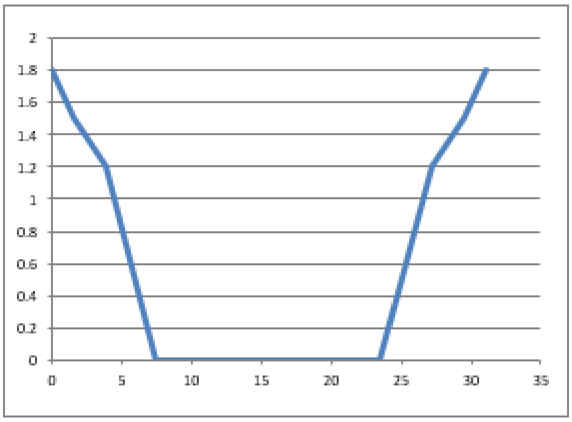
## Dewatering Area

| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 5 years = | 578.42 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 1156.84 | $m^2$ | |

**Trapezoid Calculator**

| | | | |
|---|---|---|---|
| Total Depth | **1.8** | m | |
| Extended Detention | 0.30 | 1 in | 5 |
| Depth of 1:8 | 0.3 | 1 in | 8 |
| Depth of 1:3 | 1.20 | 1 in | 3 |
| NWL Length | 39.24 | m | |
| NWL Width | 28.03 | m | |
| **NWL Area** | **1100.00** | **$m^2$** | |

| | |
|---|---|
| 0 | 1.8 |
| 1.5 | 1.50 |
| 3.9 | 1.20 |
| 7.5 | 0 |
| 23.53 | 0 |
| 27.13 | 1.2 |
| 29.53 | 1.5 |
| 31.03 | 1.8 |




**AREA**

| | | |
|---|---|---|
| Bottom Length | 27.24 | m |
| Bottom Width | 16.03 | m |
| Bottom Area | 436.72 | $m^2$ |
| Bench Length | 34.44 | m |
| Bench Width | 23.23 | m |
| Bench Area | 800.13 | $m^2$ |
| NWL Length | 39.24 | m |
| NWL Width | 28.03 | m |
| NWL Area | 1100.0 | $m^2$ |
| TED Length | 42.24 | m |
| TED Width | 31.03 | m |
| TED Area | 1310.82 | $m^2$ |

**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| **Stage** | **Storage (** | **Area (m2)** | **Length** | **Width** |
|---|---|---|---|---|
| 0 | 0.0 | 436.7 | 27.24 | 16.03 |
| 0.1 | 45.0 | 463.0 | 27.84 | 16.63 |
| 0.2 | 92.6 | 490.1 | 28.44 | 17.23 |
| 0.3 | 143.0 | 517.9 | 29.04 | 17.83 |
| 0.4 | 196.2 | 546.3 | 29.64 | 18.43 |
| 0.5 | 252.3 | 575.5 | 30.24 | 19.03 |
| 0.6 | 311.3 | 605.5 | 30.84 | 19.63 |
| 0.7 | 373.3 | 636.1 | 31.44 | 20.23 |
| 0.8 | 438.5 | 667.5 | 32.04 | 20.83 |
| 0.9 | 506.8 | 699.6 | 32.64 | 21.43 |
| 1 | 578.4 | 732.4 | 33.24 | 22.03 |
| 1.1 | 653.3 | 765.9 | 33.84 | 22.63 |
| 1.2 | 731.6 | 800.1 | 34.44 | 23.23 |
| 1.3 | 813.3 | 895.0 | 36.04 | 24.83 |
| 1.4 | 904.7 | 994.9 | 37.64 | 26.43 |
| 1.5 | 1006.1 | 1100.0 | 39.24 | 28.03 |
| 1.6 | 1118.1 | 1168.3 | 40.24 | 29.03 |
| 1.7 | 1237.0 | 1238.5 | 41.24 | 30.03 |
| 1.8 | 1363.0 | 1310.8 | 42.24 | 31.03 |

# D-2 Outfall 2

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential dewlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| $d^*$ = | 1 | m | (lower of 1 m and dp) |
| $\frac{(d_e+d_p)}{(d_e+d^*)}$ = | 1.38 | | |
| Q = | 0.40 | $m^3/s$ | 1 EY flows from RORB modellin; equivalent to Outfall 6 which has the same area |
| A = | **400** | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $\frac{V_s}{Q/A}$ = | 11.00 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

R = **95.07%**

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

### Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 11.3 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3$/ha/yr | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3$/ha/yr | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 118.77 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3)}{\text{ActualBasinVolume } (m^3)}$ 0.19 per year

Clean out every 5.26 years

***Try to minimise cleanouts - ideally, once every 5 years***

### Dewatering Area

| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 5 years = | 118.77 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 237.55 | $m^2$ | |

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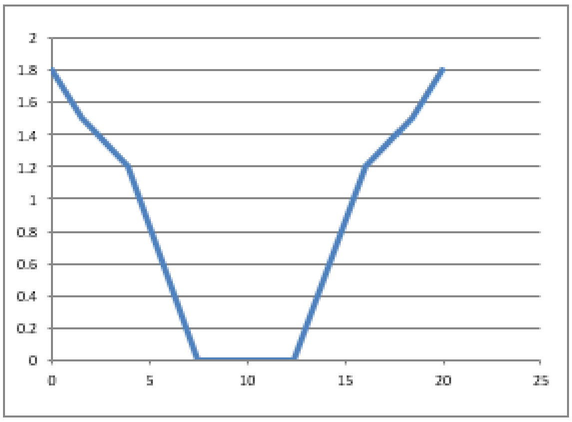
**Trapezoid Calculator**

| | | | | | |
|---|---|---|---|---|---|
| Total Depth | 1.8 | m | | | |
| Extended Detention | 0.30 | 1 in | 5 | 0 | 1.8 |
| Depth of 1:8 | 0.3 | 1 in | 8 | 1.5 | 1.50 |
| Depth of 1:3 | 1.20 | 1 in | 3 | 3.9 | 1.20 |
| NWL Length | 23.66 | m | | 7.5 | 0 |
| NWL Width | 16.90 | m | | 12.40 | 0 |
| **NWL Area** | **400.00** | **m²** | | 16.00 | 1.2 |
| | | | | 18.40 | 1.5 |
| | | | | 19.90 | 1.8 |

**AREA**

| | | |
|---|---|---|
| Bottom Length | 11.66 | m |
| Bottom Width | 4.90 | m |
| Bottom Area | 57.19 | m² |
| Bench Length | 18.86 | m |
| Bench Width | 12.10 | m |
| Bench Area | 228.32 | m² |
| NWL Length | 23.66 | m |
| NWL Width | 16.90 | m |
| NWL Area | 400.0 | m² |
| TED Length | 26.66 | m |
| TED Width | 19.90 | m |
| TED Area | 530.70 | m² |




**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| Stage | Storage ( | Area (m2) | Length | Width |
|---|---|---|---|---|
| 0 | 0.0 | 57.2 | 11.66 | 4.90 |
| 0.1 | 6.2 | 67.5 | 12.26 | 5.50 |
| 0.2 | 13.5 | 78.5 | 12.86 | 6.10 |
| 0.3 | 21.9 | 90.3 | 13.46 | 6.70 |
| 0.4 | 31.5 | 102.7 | 14.06 | 7.30 |
| 0.5 | 42.5 | 115.9 | 14.66 | 7.90 |
| 0.6 | 54.7 | 129.8 | 15.26 | 8.50 |
| 0.7 | 68.4 | 144.4 | 15.86 | 9.10 |
| 0.8 | 83.6 | 159.8 | 16.46 | 9.70 |
| 0.9 | 100.4 | 175.8 | 17.06 | 10.30 |
| 1 | 118.8 | 192.6 | 17.66 | 10.90 |
| 1.1 | 138.9 | 210.1 | 18.26 | 11.50 |
| 1.2 | 160.8 | 228.3 | 18.86 | 12.10 |
| 1.3 | 184.6 | 280.4 | 20.46 | 13.70 |
| 1.4 | 213.6 | 337.7 | 22.06 | 15.30 |
| 1.5 | 248.5 | 400.0 | 23.66 | 16.90 |
| 1.6 | 289.7 | 441.6 | 24.66 | 17.90 |
| 1.7 | 335.2 | 485.1 | 25.66 | 18.90 |
| 1.8 | 385.0 | 530.7 | 26.66 | 19.90 |

# D-3 Outfall 3

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential devlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| d* = | 1 | m | (lower of 1 m and dp) |
| $(d_e+d_p)$ / $(d_e+d^*)$ = | 1.38 | | |
| Q = | 0.41 | $m^3/s$ | 1 EY flows from RORB modelling |
| A = | 450 | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $V_s$ / Q/A = | 12.22 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

**R = 95.58%**

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

### Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 14.6 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3$/ha/yr | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3$/ha/yr | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 146.63 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = Catchment Load ($m^3$) / ActualBasinVolume ($m^3$) = 0.20 per year

Clean out every 5.01 years

***Try to minimise cleanouts - ideally, once every 5 years***

## Dewatering Area

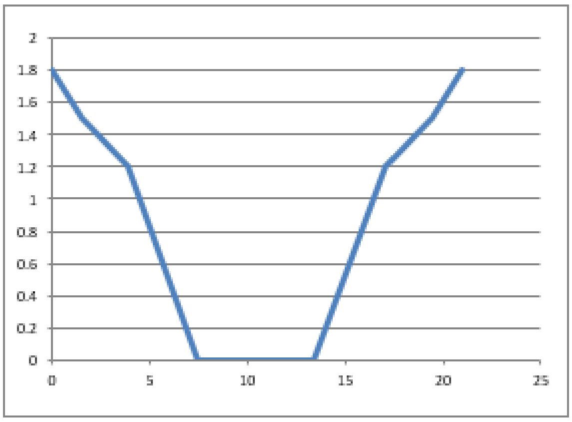
| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 5 years = | 146.63 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 293.25 | $m^2$ | |

**Trapezoid Calculator**

| | | | | | |
|---|---|---|---|---|---|
| Total Depth | **1.8** | m | | | |
| Extended Detention | 0.30 | 1 in | 5 | 0 | 1.8 |
| Depth of 1:8 | 0.3 | 1 in | 8 | 1.5 | 1.50 |
| Depth of 1:3 | 1.20 | 1 in | 3 | 3.9 | 1.20 |
| NWL Length | 25.10 | m | | 7.5 | 0 |
| NWL Width | 17.93 | m | | 13.43 | 0 |
| **NWL Area** | **450.00** | **m²** | | 17.03 | 1.2 |
| | | | | 19.43 | 1.5 |
| | | | | 20.93 | 1.8 |

**AREA**

| | | |
|---|---|---|
| Bottom Length | 13.10 | m |
| Bottom Width | 5.93 | m |
| Bottom Area | 77.66 | m² |
| Bench Length | 20.30 | m |
| Bench Width | 13.13 | m |
| Bench Area | 266.50 | m² |
| NWL Length | 25.10 | m |
| NWL Width | 17.93 | m |
| NWL Area | 450.0 | m² |
| TED Length | 28.10 | m |
| TED Width | 20.93 | m |
| TED Area | 588.08 | m² |




**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| **Stage** | **Storage (** | **Area (m2)** | **Length** | **Width** |
|---|---|---|---|---|
| 0 | 0.0 | 77.7 | 13.10 | 5.93 |
| 0.1 | 8.3 | 89.4 | 13.70 | 6.53 |
| 0.2 | 17.9 | 101.9 | 14.30 | 7.13 |
| 0.3 | 28.7 | 115.2 | 14.90 | 7.73 |
| 0.4 | 40.9 | 129.1 | 15.50 | 8.33 |
| 0.5 | 54.5 | 143.7 | 16.10 | 8.93 |
| 0.6 | 69.7 | 159.1 | 16.70 | 9.53 |
| 0.7 | 86.4 | 175.2 | 17.30 | 10.13 |
| 0.8 | 104.7 | 192.0 | 17.90 | 10.73 |
| 0.9 | 124.8 | 209.6 | 18.50 | 11.33 |
| 1 | 146.6 | 227.8 | 19.10 | 11.93 |
| 1.1 | 170.3 | 246.8 | 19.70 | 12.53 |
| 1.2 | 196.0 | 266.5 | 20.30 | 13.13 |
| 1.3 | 223.6 | 322.5 | 21.90 | 14.73 |
| 1.4 | 257.0 | 383.7 | 23.50 | 16.33 |
| 1.5 | 296.6 | 450.0 | 25.10 | 17.93 |
| 1.6 | 342.9 | 494.0 | 26.10 | 18.93 |
| 1.7 | 393.6 | 540.1 | 27.10 | 19.93 |
| 1.8 | 449.0 | 588.1 | 28.10 | 20.93 |

# D-4 Outfall 4

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential devlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| $d^*$ = | 1 | m | (lower of 1 m and dp) |
| $\frac{(d_e+d_p)}{(d_e+d^*)}$ = | 1.38 | | |
| Q = | 0.54 | $m^3/s$ | 1 EY flows from RORB modelling |
| A = | **600** | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $\frac{V_s}{Q/A}$ = | 12.30 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

**R = 95.61%**

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

### Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 17.8 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3/ha/yr$ | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3/ha/yr$ | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 236.72 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3)}{\text{ActualBasinVolume } (m^3)}$ 0.15 per year

Clean out every 6.64 years

***Try to minimise cleanouts - ideally, once every 5 years***

### Dewatering Area

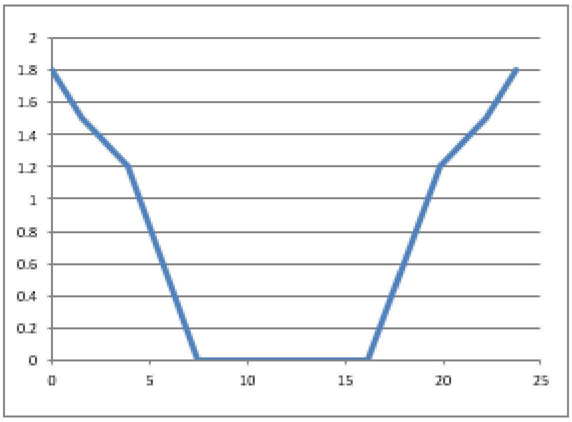
| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 7 years = | 236.72 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 473.44 | $m^2$ | |

**Trapezoid Calculator**

| | | | | | |
|---|---|---|---|---|---|
| Total Depth | 1.8 | m | | | |
| Extended Detention | 0.30 | 1 in | 5 | 0 | 1.8 |
| Depth of 1:8 | 0.3 | 1 in | 8 | 1.5 | 1.5 |
| Depth of 1:3 | 1.20 | 1 in | 3 | 3.9 | 1.2 |
| NWL Length | 28.98 | m | | 7.5 | 0 |
| NWL Width | 20.70 | m | | 16.20 | 0 |
| **NWL Area** | **600.00** | **$m^2$** | | 19.80 | 1.2 |
| | | | | 22.20 | 1.5 |
| | | | | 23.70 | 1.8 |

**AREA**

| | | |
|---|---|---|
| Bottom Length | 16.98 | m |
| Bottom Width | 8.70 | m |
| Bottom Area | 147.78 | $m^2$ |
| Bench Length | 24.18 | m |
| Bench Width | 15.90 | m |
| Bench Area | 384.55 | $m^2$ |
| NWL Length | 28.98 | m |
| NWL Width | 20.70 | m |
| NWL Area | 600.0 | $m^2$ |
| TED Length | 31.98 | m |
| TED Width | 23.70 | m |
| TED Area | 758.05 | $m^2$ |




**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| Stage | Storage ( | Area (m2) | Length | Width |
|---|---|---|---|---|
| 0 | 0.0 | 147.8 | 16.98 | 8.70 |
| 0.1 | 15.5 | 163.6 | 17.58 | 9.30 |
| 0.2 | 32.7 | 180.0 | 18.18 | 9.90 |
| 0.3 | 51.6 | 197.3 | 18.78 | 10.50 |
| 0.4 | 72.2 | 215.2 | 19.38 | 11.10 |
| 0.5 | 94.6 | 233.8 | 19.98 | 11.70 |
| 0.6 | 118.9 | 253.2 | 20.58 | 12.30 |
| 0.7 | 145.2 | 273.3 | 21.18 | 12.90 |
| 0.8 | 173.6 | 294.1 | 21.78 | 13.50 |
| 0.9 | 204.1 | 315.6 | 22.38 | 14.10 |
| 1 | 236.7 | 337.9 | 22.98 | 14.70 |
| 1.1 | 271.6 | 360.9 | 23.58 | 15.30 |
| 1.2 | 308.9 | 384.6 | 24.18 | 15.90 |
| 1.3 | 348.5 | 451.2 | 25.78 | 17.50 |
| 1.4 | 395.0 | 523.1 | 27.38 | 19.10 |
| 1.5 | 448.7 | 600.0 | 28.98 | 20.70 |
| 1.6 | 510.2 | 650.7 | 29.98 | 21.70 |
| 1.7 | 576.8 | 703.4 | 30.98 | 22.70 |
| 1.8 | 648.7 | 758.1 | 31.98 | 23.70 |

# D-5 Outfall 5

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential devlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for NW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| $d^*$ = | 1 | m | (lower of 1 m and dp) |
| $\frac{(d_e+d_p)}{(d_e+d^*)}$ = | 1.38 | | |
| Q = | 0.27 | $m^3/s$ | 1 EY flows from RORB modelling |
| A = | **350** | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $\frac{V_s}{Q/A}$ = | 14.28 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

| | |
|---|---|
| R = | **96.25%** |

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

### Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 8.3 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3$/ha/yr | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3$/ha/yr | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 92.35 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3)}{\text{ActualBasinVolume } (m^3)}$ 0.18 per year

Clean out every 5.58 years

***Try to minimise cleanouts - ideally, once every 5 years***

### Dewatering Area

| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for NW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 6 years = | 92.35 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 184.71 | $m^2$ | |

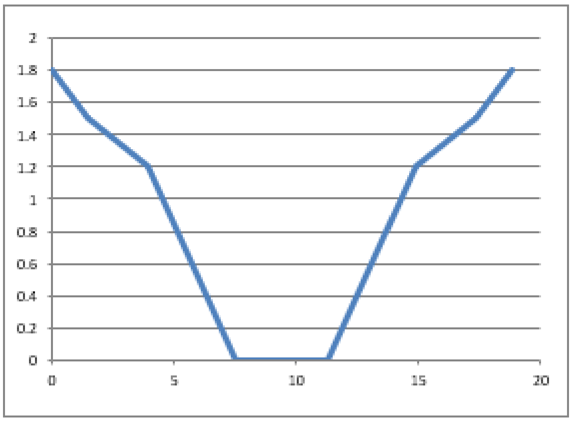
## Trapezoid Calculator

| | | | |
|---|---|---|---|
| Total Depth | **1.8** | m | |
| Extended Detention | 0.30 | 1 in | 5 |
| Depth of 1:8 | 0.3 | 1 in | 8 |
| Depth of 1:3 | 1.20 | 1 in | 3 |
| NWL Length | 22.14 | m | |
| NWL Width | 15.81 | m | |
| **NWL Area** | **350.00** | **$m^2$** | |

| | |
|---|---|
| 0 | 1.8 |
| 1.5 | 1.5 |
| 3.9 | 1.2 |
| 7.5 | 0 |
| 11.31 | 0 |
| 14.91 | 1.2 |
| 17.31 | 1.5 |
| 18.81 | 1.8 |

### AREA

| | | |
|---|---|---|
| Bottom Length | 10.14 | m |
| Bottom Width | 3.81 | m |
| Bottom Area | 38.63 | $m^2$ |
| Bench Length | 17.34 | m |
| Bench Width | 11.01 | m |
| Bench Area | 190.89 | $m^2$ |
| NWL Length | 22.14 | m |
| NWL Width | 15.81 | m |
| NWL Area | 350.0 | $m^2$ |
| TED Length | 25.14 | m |
| TED Width | 18.81 | m |
| TED Area | 472.84 | $m^2$ |




### STORAGE

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| Stage | Storage ( | Area (m2) | Length | Width |
|---|---|---|---|---|
| 0 | 0.0 | 38.6 | 10.14 | 3.81 |
| 0.1 | 4.3 | 47.4 | 10.74 | 4.41 |
| 0.2 | 9.5 | 56.8 | 11.34 | 5.01 |
| 0.3 | 15.6 | 67.0 | 11.94 | 5.61 |
| 0.4 | 22.9 | 77.9 | 12.54 | 6.21 |
| 0.5 | 31.2 | 89.5 | 13.14 | 6.81 |
| 0.6 | 40.8 | 101.8 | 13.74 | 7.41 |
| 0.7 | 51.6 | 114.9 | 14.34 | 8.01 |
| 0.8 | 63.7 | 128.6 | 14.94 | 8.61 |
| 0.9 | 77.3 | 143.1 | 15.54 | 9.21 |
| 1 | 92.4 | 158.3 | 16.14 | 9.81 |
| 1.1 | 109.0 | 174.2 | 16.74 | 10.41 |
| 1.2 | 127.2 | 190.9 | 17.34 | 11.01 |
| 1.3 | 147.1 | 238.8 | 18.94 | 12.61 |
| 1.4 | 172.0 | 291.8 | 20.54 | 14.21 |
| 1.5 | 202.2 | 350.0 | 22.14 | 15.81 |
| 1.6 | 238.3 | 388.9 | 23.14 | 16.81 |
| 1.7 | 278.4 | 429.9 | 24.14 | 17.81 |
| 1.8 | 322.7 | 472.8 | 25.14 | 18.81 |

# D-6 Outfall 6

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential devlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| d* = | 1 | m | (lower of 1 m and dp) |
| $(d_e+d_p)$ / $(d_e+d^*)$ = | 1.38 | | |
| Q = | 0.39 | $m^3/s$ | 1 EY flows from RORB modelling |
| A = | **400** | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $V_s$ / Q/A = | 11.24 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

R = **95.17%**

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

## Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 11.8 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3/ha/yr$ | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3/ha/yr$ | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 118.77 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = Catchment Load ($m^3$) / ActualBasinVolume ($m^3$) = 0.20 per year

Clean out every 5.03 years

***Try to minimise cleanouts - ideally, once every 5 years***

## Dewatering Area

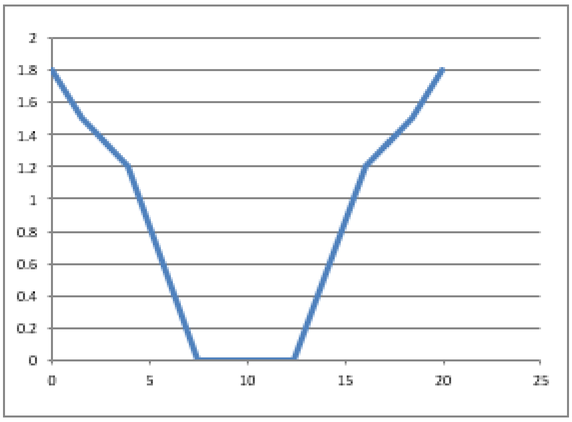
| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 5 years = | 118.77 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 237.55 | $m^2$ | |

**Trapezoid Calculator**

| | | | | | |
|---|---|---|---|---|---|
| Total Depth | 1.8 | m | | | |
| Extended Detention | 0.30 | 1 in | 5 | 0 | 1.8 |
| Depth of 1:8 | 0.3 | 1 in | 8 | 1.5 | 1.5 |
| Depth of 1:3 | 1.20 | 1 in | 3 | 3.9 | 1.2 |
| NWL Length | 23.66 | m | | 7.5 | 0 |
| NWL Width | 16.90 | m | | 12.40 | 0 |
| **NWL Area** | **400.00** | **$m^2$** | | 16.00 | 1.2 |
| | | | | 18.40 | 1.5 |
| | | | | 19.90 | 1.8 |

**AREA**

| | | |
|---|---|---|
| Bottom Length | 11.66 | m |
| Bottom Width | 4.90 | m |
| Bottom Area | 57.19 | $m^2$ |
| Bench Length | 18.86 | m |
| Bench Width | 12.10 | m |
| Bench Area | 228.32 | $m^2$ |
| NWL Length | 23.66 | m |
| NWL Width | 16.90 | m |
| NWL Area | 400.0 | $m^2$ |
| TED Length | 26.66 | m |
| TED Width | 19.90 | m |
| TED Area | 530.70 | $m^2$ |




**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| Stage | Storage ( | Area (m2) | Length | Width |
|---|---|---|---|---|
| 0 | 0.0 | 57.2 | 11.66 | 4.90 |
| 0.1 | 6.2 | 67.5 | 12.26 | 5.50 |
| 0.2 | 13.5 | 78.5 | 12.86 | 6.10 |
| 0.3 | 21.9 | 90.3 | 13.46 | 6.70 |
| 0.4 | 31.5 | 102.7 | 14.06 | 7.30 |
| 0.5 | 42.5 | 115.9 | 14.66 | 7.90 |
| 0.6 | 54.7 | 129.8 | 15.26 | 8.50 |
| 0.7 | 68.4 | 144.4 | 15.86 | 9.10 |
| 0.8 | 83.6 | 159.8 | 16.46 | 9.70 |
| 0.9 | 100.4 | 175.8 | 17.06 | 10.30 |
| 1 | 118.8 | 192.6 | 17.66 | 10.90 |
| 1.1 | 138.9 | 210.1 | 18.26 | 11.50 |
| 1.2 | 160.8 | 228.3 | 18.86 | 12.10 |
| 1.3 | 184.6 | 280.4 | 20.46 | 13.70 |
| 1.4 | 213.6 | 337.7 | 22.06 | 15.30 |
| 1.5 | 248.5 | 400.0 | 23.66 | 16.90 |
| 1.6 | 289.7 | 441.6 | 24.66 | 17.90 |
| 1.7 | 335.2 | 485.1 | 25.66 | 18.90 |
| 1.8 | 385.0 | 530.7 | 26.66 | 19.90 |

# D-7 Outfall 7

## Calculations

| | | | |
|---|---|---|---|
| ***Sediment Target =*** | **Very fine sand** | | *Very fine saand for standard residential devlopments* |
| Vs = | 0.011 | m/s | This value changes for different particle size target |
| $d_e$ = | 0.30 | m | Extended Detention Depth *max 0.35 for MW* |
| $d_p$ = | 1.5 | m | Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments* |
| d* = | 1 | m | (lower of 1 m and dp) |
| $(d_e+d_p)$ = / $(d_e+d^*)$ | 1.38 | | |
| Q = | 0.63 | $m^3/s$ | 1 EY flows from RORB modelling |
| A = | 700 | $m^2$ | Area of the sediment basin at NWL |
| L/W = | 1.4 | | Length/Width Ratio (assuming rectangular shape) |
| $V_s$ = / Q/A | 12.22 | | |
| λ = | 0.11 | | Pond shape assumption (see figure 10.5 above) |
| n = | 1.12 | | |

**Fraction of Initial Solids Removed**

| | |
|---|---|
| R = | **95.58%** |

***Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle***

### Cleanout Frequency

| | | | |
|---|---|---|---|
| Catchment area = | 27.2 | ha | Just urban catchment |
| Sediment load = | 1.60 | $m^3$/ha/yr | 1.6 - Willing and Partners 1992 - urban load |
| Gross Pollutant Load = | 0.40 | $m^3$/ha/yr | 0.4 - Alison et al 1998 |

***Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***

| | | | |
|---|---|---|---|
| Actual basin depth = | 1.00 | m | 500 mm below the NWL (i.e. dp-0.5) |
| Actual Basin volume = | 300.89 | $m^3$ | Basin volume at 500 mm below the NWL |

Therefore, cleanout frequency required = Catchment Load ($m^3$) / ActualBasinVolume ($m^3$) 0.18 per year — Clean out every 5.53 years

***Try to minimise cleanouts - ideally, once every 5 years***

### Dewatering Area

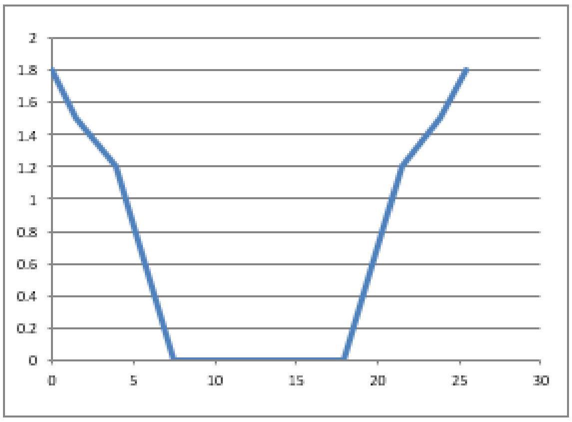
| | | | |
|---|---|---|---|
| Dewatering depth = | 0.50 | m | Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils* |
| Sediment volume collected every 6 years = | 300.89 | $m^3$ | volume of sediment acumulated up to 0.5 m below the NWL |
| Required Dewatering area = | 601.78 | $m^2$ | |

**Trapezoid Calculator**

| | | | | | |
|---|---|---|---|---|---|
| Total Depth | **1.8** | m | | | |
| Extended Detention | 0.30 | 1 in | 5 | 0 | 1.8 |
| Depth of 1:8 | 0.3 | 1 in | 8 | 1.5 | 1.5 |
| Depth of 1:3 | 1.20 | 1 in | 3 | 3.9 | 1.2 |
| NWL Length | 31.30 | m | | 7.5 | 0 |
| NWL Width | 22.36 | m | | 17.86 | 0 |
| **NWL Area** | **700.00** | **$m^2$** | | 21.46 | 1.2 |
| | | | | 23.86 | 1.5 |
| | | | | 25.36 | 1.8 |

**AREA**

| | | |
|---|---|---|
| Bottom Length | 19.30 | m |
| Bottom Width | 10.36 | m |
| Bottom Area | 200.01 | $m^2$ |
| Bench Length | 26.50 | m |
| Bench Width | 17.56 | m |
| Bench Area | 465.44 | $m^2$ |
| NWL Length | 31.30 | m |
| NWL Width | 22.36 | m |
| NWL Area | 700.0 | $m^2$ |
| TED Length | 34.30 | m |
| TED Width | 25.36 | m |
| TED Area | 870.00 | $m^2$ |




**STORAGE**

| | |
|---|---|
| Increments | 0.1 |
| Side length per increment 1:3 | 0.3 |
| Side length per increment 1:8 | 0.8 |
| Side length per increment 1:5 | 0.5 |

| **Stage** | **Storage (** | **Area (m2)** | **Length** | **Width** |
|---|---|---|---|---|
| 0 | 0.0 | 200.0 | 19.30 | 10.36 |
| 0.1 | 20.9 | 218.2 | 19.90 | 10.96 |
| 0.2 | 43.6 | 237.1 | 20.50 | 11.56 |
| 0.3 | 68.3 | 256.7 | 21.10 | 12.16 |
| 0.4 | 95.0 | 277.0 | 21.70 | 12.76 |
| 0.5 | 123.7 | 298.0 | 22.30 | 13.36 |
| 0.6 | 154.6 | 319.8 | 22.90 | 13.96 |
| 0.7 | 187.6 | 342.2 | 23.50 | 14.56 |
| 0.8 | 223.0 | 365.4 | 24.10 | 15.16 |
| 0.9 | 260.7 | 389.4 | 24.70 | 15.76 |
| 1 | 300.9 | 414.0 | 25.30 | 16.36 |
| 1.1 | 343.5 | 439.4 | 25.90 | 16.96 |
| 1.2 | 388.8 | 465.4 | 26.50 | 17.56 |
| 1.3 | 436.6 | 538.5 | 28.10 | 19.16 |
| 1.4 | 491.9 | 616.7 | 29.70 | 20.76 |
| 1.5 | 555.1 | 700.0 | 31.30 | 22.36 |
| 1.6 | 626.7 | 754.7 | 32.30 | 23.36 |
| 1.7 | 703.8 | 811.3 | 33.30 | 24.36 |
| 1.8 | 786.7 | 870.0 | 34.30 | 25.36 |

# APPENDIX E
# LOT-SCALE RAINWAER HARVESTING CALCULATION

This section provides a summary of rainwater harvesting calculations.

## E-1 Catchment

- Net developable area = 88.82 ha (Outfall 1, 2, 5 and 6 extracted from Jetty Road Urban Growth Plan)
- Lot density = 15 lots/ha
- No. of lots = 1,160
- Average log size = 400 $m^2$
- % of roof area in each lot = 60% (Planning Practice Note 27 | Understanding the Residential Development Standards (ResCode))
- % of roof connected to rainwater tank = 50%
- Roof catchment = 60% x 50% x 1160 x 400 $m^2$ = 13.92 ha

## E-2 Reuse Demand

- Individual tank volume = 2 kL
- No. of tanks = 1160
- Reuse demand type = daily demand
- Reuse application = toilet flushing
- Reuse demand per person = 20 L/day (Melbourne Water MUSIC modelling guidelines, 2018)
- Average household size = 2.6 persons/dwelling (Jetty Road Urban Growth Plan)
- Total reuse demand = 1160 x 2.6 x 20 L/day = 60.32 kL/day

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